VOL.107 NO.IR2. JUNE 1981

JOURNAL OF THE IRRIGATION AND DRAINAGE DIVISION

PROCEEDINGS OF
THE AMERICAN SOCIETY
OF CIVIL ENGINEERS

THIS ISSUE INCLUDES
FIVE PAPERS ON
LAND SUSIDENCE

VOL.107 NO.IR2. JUNE 1981

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PROCEEDINGS OF
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OF CIVIL ENGINEERS

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ISSN 0044-7978

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FIVE PAPERS ON
LAND SUSIDENCE

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This Journal is published quarterly by the American Society of Civil Engineers. Publications office is at 345 East 47th Street, New York, N.Y. 10017. Address all ASCE correspondence to the Editorial and General Offices at 345 East 47th Street, New York, N.Y. 10017. Allow six weeks for change of address to become effective. Subscription price to members is $7.50. Nonmember subscriptions available; prices obtainable on request. Second-class postage paid at New York, N.Y. and at additional mailing offices. IR.

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INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

*Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16329 SUBSIDENCE DUE TO GROUND-WATER WITHDRAWAL

KEY WORDS: **Aquifers;** Aquitards; **Compaction (soils);** Confined water; Geology; **Ground water; Land subsidence;** Porosity; Storage capacity; **Subsidence**

ABSTRACT: Land subsidence due to ground-water withdrawal in the United States is summarized. The characteristic geologic environment for subsidence due to ground-water withdrawal is that of young unconsolidated sediments of high porosity laid down in alluvial, lacustrine, or shallow marine environments. All areas are underlain by semiconfined or confined aquifer systems containing aquifers of sand or gravel, or both, of low compressibility interbedded with clayey aquitards of low vertical permeability and high compressibility under virgin stresses. Subsidence ranges from 0.3 m at Savannah, Georgia, to 9.0 m in the San Joaquin Valley, California. The volume of subsidence in the San Joaquin Valley is very large—about 19,250 hm^3. The usable storage capacity of the aquifer system, defined as the volume that can be taken from or recharged to the system, is not changed appreciably by the compaction of the aquitards but the specific storage is greatly reduced for later cycles of water-level decline through the same pore pressure range.

REFERENCE: Poland, Joseph F., "Subsidence in United States Due to Ground-Water Withdrawal," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16329,** June, 1981, pp. 115-135

16331 LEGAL ASPECTS OF SUBSIDENCE DUE TO WELL PUMPING

KEY WORDS: **Ground water;** Groundwater depletion; Groundwater hydraulics; Groundwater lowering; **Hydrology; Land subsidence; Law (jurisprudence);** Legal factors; **Liabilities; Subsidence;** Water management (applied); **Water rights; Well pumps**

ABSTRACT: Subsidence of land due to pumping of ground water is a relatively recent phenomenon. Early cases based on English common law held a person who pumped ground water was not liable for damage to a neighbor's property. In general, the attitude of the nation toward ground-water pumping and subsidence was one of laissez faire. Recent cases and legislative discussions indicate a trend toward holding ground-water pumpers liable for adverse effects of their pumping. The growing recognition that water is a limited resources will, in time, bring about mandatory ground-water management which, in turn, will place the burden of subsidence control on local water agencies.

REFERENCE: Kopper, William, and Finlayson, Donald, "Legal Aspects of Subsidence Due to Well Pumping," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16331,** June, 1981, pp. 137-149

16335 ECONOMIC CONSEQUENCES OF LAND SURFACE SUBSIDENCE

KEY WORDS: Economic factors; **Economic impact; Ground water; Groundwater depletion;** Groundwater hydrology; **Ground water recharge; Land subsidence; Overdraft;** Subsidence; Time factors; Water resources management

ABSTRACT: Overdraft in the Santa Clara Valley ground-water basin caused land surface subsidence over an area of about 63,000 hectares with a maximum depression of about 3.6 meters between 1912 and 1967. Since cessation of overdraft and replenishment of ground-water levels in 1969, there has been no significant land surface subsidence. During the period of active land surface subsidence the following occurred: water well casings buckled; sanitary and storm drainage sewers lost capacity as a result of changes in slope; roads and railroads had to be raised to stay above floodwaters; and new levees had to be erected and old ones raised to protect developed areas against flooding from San Franciso Bay. These damages resulting from land surface subsidence are estimated to have amounted to over $130,000,000.

REFERENCE: Fowler, Lloyd C., "Economic Consequences of Land Surface Subsidence," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16335,** June, 1981, pp. 151-159

16339 LAND SUBSIDENCE IN TWO TEXAS COUNTIES

KEY WORDS: Clay soils; Governmental role; **Ground water;** History; **Land subsidence;** State government; Surface waters; **Texas;** Time factors; **Water levels**

ABSTRACT: Land-surface subsidence is of critical concern for the Harris-Galveston Region of Texas. Subsidence has been predominately associated with the removal of ground water in this area. Water-level and pumpage information is available from the turn of the century for defined areas within the region. This information has been used along with leveling data to show the rates, amounts and possible causes of subsidence within these areas. Because of the continued subsidence, the Texas State Legislature created the Harris-Galveston Coastal Subsidence District. The district's primary responsibility is to halt subsidence. The rate of subsidence within this region has decreased significantly in several areas with the introduction of surface water.

REFERENCE: Neighbors, Ronald J., "Subsidence in Harris and Galveston Counties, Texas," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16339**, June, 1981, pp. 161-174

16343 INDUCED SINKHOLES: AN ENGINEERING PROBLEM

KEY WORDS: Collapse; Damage; **Drainage; Ground water;** Groundwater depletion; Human factors; **Hydrology; Land subsidence; Pumping; Sinkholes; Subsidence;** Water table

ABSTRACT: Induced sinkholes are those related to man's activities, whereas natural sinkholes are not. Induced sinkholes are divided into two types: those resulting from a decline in the water table due to ground-water withdrawals and those resulting from construction. Almost all induced sinkholes occur where cavities develop in residual or other unconsolidated deposits overlying openings in carbonate rocks. The downward migration of the deposits into underlying openings in bedrock and the formation and collapse of resulting cavities are caused or accelerated by a decline in the water table that results in: (1) Loss of buoyant support; (2) increase in the velocity of movement of water; (3) water-level fluctuations at the base of unconsolidated deposits; and (4) induced recharge. Most sinkholes resulting from construction are due to the diversion of drainage over openings in bedrock.

REFERENCE: Newton, John G., "Induced Sinkholes: An Engineering Problem," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16343**, June, 1981, pp. 175-185

16323 MODEL OF GROUNDWATER FLOW

KEY WORDS: Calculators; **Computation; Equations; Ground water; Groundwater flow; Mathematical programming;** Nonlinear systems; Numerical calculations; Seepage

ABSTRACT: The subdomain numerical integration method is applied to the nonlinear one-dimensional saturated groundwater flow equation (Boussinesq equation). Because the state variable of the resulting numerical analog can be cast in terms of spatial coordinates, nonlinearity can be eliminated, greatly increasing accuracy and simplifying the numerical code. As a result, the solution can be performed on hand held programmable calculators. Good results were obtained when applying the method to confined and unconfined (nonlinear) aquifer examples.

REFERENCE: Hromadka, T. V., II, and Guymon, G. L., "Subdomain Integration Model of Ground-Water Flow," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16323**, June, 1981, pp. 187-195

16350 PVC AND ABS WELL CASING JOINTS

KEY WORDS: **Irrigation;** Joints; **Plastic properties; Plastics;** Stresses; Subsurface structures; Well casings; **Wells**

ABSTRACT: An experimental test program was conducted to determine the parameters influencing the short-term and long-term axial strength of polyvinyl chloride (PVC) and acrylonitrile-butadiene-styrene (ABS) well casing joints. Joint configurations tested include: bell couplings, slip couplings, male/female threaded couplings, and threaded casing. The effect of cure time and cure environment on solvent weld joints, torque on threaded casings, and time of load application on joint axial strength were evaluated. from axial creep strength data, a regression analysis was conducted to calculate proposed design stresses for thermoplastic well casing systems. Based on these design stresses, the commercially available thermoplastic materials and joint systems tested had adequate strength for dry string lengths for over 1,000 feet (300 m). The effects of a simulated blockage test were also evaluated.

REFERENCE: Kurt, Carl E., and Pate, W. Denney, "Tensile Strength of PVC and ABS Well Casing Joints," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16350**, June, 1981, pp. 197-208

16342 CALIFORNIA IRRIGATION RETURN FLOW

KEY WORDS: **California; Dissolved solids;** Drainage; Drainage patterns; **Drainage water; Irrigation effects;** Irrigation practices; **Nitrates;** Nutrients; **Return flow;** Salinity; Sediments; **Suspended solids; Water quality; Water quality control;** Water quality standards

ABSTRACT: The variability in the quantity and quality of surface irrigation return flows from two representative California irrigation projects are reported. The two projects are: (1)The 66,420 ha Glenn-Colusa Irrigation District in the Sacramento Valley, and (2)the 17,820 ha Panoche Drainage District for 1975, a typical irrigation season. The variations noted are attributed to differences in water application methods, crops, soil chemical and physical properties. The impacts of return flows to the Sacramento and San Joaquin Rivers are appraised and examined relative to the ongoing Section 208 Water Quality Management Planning efforts.

REFERENCE: Tanji, Kenneth K., "California Irrigation Return Flow Case Studies," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16342**, June, 1981, pp. 209-220

16345 GRAND VALLEY IRRIGATION RETURN FLOW

KEY WORDS: **Colorado;** Colorado River; **Irrigation;** Irrigation effects; Irrigation water; **Legislation; Return flow; Salinity; Water quality;** Water quality control; Water quality standards; Water resources

ABSTRACT: Irrigation water supply is furnished annually to about 71,500 acres (28,932 ha) of land in the Grand Valley of western Colorado. Return flows from that irrigation contribute about 780,000 tons (708×10^6 kg) of salt to the Colorado River each year, causing an increase of 77 mg/L in the salinity concentration at Imperial Dam. This case study for the ASCE Task Committee on Water Quality Programs Affecting Agricultural Return Flows in Arid Areas is centered around three areas: (1)Physical and water quality data for irrigation and return flows in the valley; (2)identification of regulations that affect this irrigation and its return flows; and (3)the proposed program for controlling salinity contributions from the valley to the Colorado River. With water quality standards that have been established for the Colorado River and the continued development of water resources in the basin, a salinity control program for Grand Valley is needed.

REFERENCE: Keys, John W., III, "Grand Valley Irrigation Return Flow Case Study," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16345**, June, 1981, pp. 221-232

16341 WATER QUALITY AND IMPERIAL VALLEY DRAINAGE

KEY WORDS: **California; Drainage effects; Drainage practices;** Drainage systems; Irrigated land; Irrigation; **Irrigation effects;** Irrigation practices; **Management planning;** Project management; **Salinity; Water management (applied); Water quality control;** Water sampling

ABSTRACT: The Federal Water Pollution Control Act (FWPCA) mandates the development of best management practices for drainage flows from irrigated lands. Data on existing irrigation and drainage practices in California's Imperial Valley have been collected and analyzed. These data indicate that the dissolved salts in the drainage water originate in the imported irrigation water, and that the drainage water's salinity is inversely related to irrigation efficiency. Located in a closed basin, the drainage in the Imperial Valley Flows into the basin's sump, the Salton Sea, and continued crop production is dependent upon the maintenance of the Sea as a sump for the Valley. For the Imperial Valley, Salinity Control of the drainage flows should not be an objective of a best management practices plan.

REFERENCE: Valantine, Vernon E., "Water Quality Controls on Imperial Valley Drainage," *Journal of the Irrigation and Drainage Division,* ASCE, Vol. 107, No. IR2, **Proc. Paper 16341**, June, 1981, pp. 233-237

Foreword to Five Papers on Land Subsidence

By Arnold I. Johnson,[1] F. ASCE

Subsidence, or land-surface sinking, to a large extent is a man-induced natural hazard. It results primarily from the withdrawal of water, oil, or gas from subsurface zones, the compaction of sediments, the shrinking and oxidation of organic deposits, the withdrawal of steam for geothermal power, or the extractions of solids by mining. It can be the type that occurs subtly and slowly over large areas in localities of unconsolidated sediments or the type that occurs locally and dramatically through collapse in karst terrain or mining areas. Unfortunately, a good part of the known subsidence is of the first type and thus occurs so slowly that people don't become concerned until the situation is essentially of crisis proportions.

Many areas of subsidence are known throughout the world, a number of the major ones being in the U.S. It seems logical to predict that new subsidence will multiply many fold in the next few decades as a result of accelerated exploitation of natural resources, especially ground water or petroleum, in order to meet the water supply and energy demands of the increasing population and industrial development throughout the world. Unfortunately, many planners of industrial complexes, urban developments, and water-resources systems are not adequately informed about the potential hazards and problems that can result from land subsidence due to heavy withdrawal of water, gas, or oil from subsurface zones.

Most areas of known subsidence are along coasts of water bodies where it becomes quite obvious when the ocean or lake starts moving further up on the shore. In some such areas, the usual heavy population and intensive industrial development are protected from inundation by many feet of water only by construction of an extensive system of dikes, flood walls, locks, and pumping stations costing as much as millions of dollars.

The following group of papers was presented at a session sponsored by the Committee on Ground Water, Irrigation and Drainage Division, at the ASCE Annual Convention, held in Atlanta, Georgia in October 1979. The papers were updated subsequently for this publication. The purpose of this series was to inform as many people as possible about the complicated legal, socio-economic, and environmental effects resulting from subsidence caused by heavy withdrawal of ground water.

The first paper, by Joseph F. Poland, serves as the keynote paper for the group. The paper describes the subsidence phenomenon and describes 17 areas of major subsidence in the United States. The author, Dr. Poland, has spent

[1]Chairman, Committee on Ground Water; Consultant, Woodward-Clyde Consultants, Denver, CO.

nearly 40 yr on subsidence research and it is only fitting that this series of papers honor his pioneering study and dedicated investigation of this phenomenon.

The authors of the second paper are William Kopper and D. J. Finlayson. They first summarize the legal note of the nation's *laissez faire* attitude toward ground-water pumpage and subsidence and then describe some of the important legal precedents that indicate a shift in policy toward holding pumpers liable for subsidence. In some recent cases it has been held that land ownership creates a duty to use reasonable care in the exercise of property rights.

The third paper, by Lloyd C. Fowler, describes damage resulting from subsidence in an estimated amount of $100,000,000. Ground-water overdraft in the Santa Clara Valley, at the south end of the San Francisco Bay, caused land subsidence of about 4 m (13 ft) between 1914 and 1969. The damage occurring during that period included buckling of water-well casings, lost capacity of sanitary and storm drainage sewers as a result of changes in slope, and flooding from the San Francisco Bay of roads, railroads, and developed areas, thus requiring construction of levees and other protective works.

Ronald J. Neighbors authored the fourth paper; he notes that excessive pumping of ground water in the Houston-Galveston area has resulted in water level declines as much as 98.5 m (325 ft) between 1943 and 1973. Subsidence in the area has resulted in permanent inundation or increased exposure to flooding of large areas, caused well casings to collapse, lessened the efficiency of storm drainage facilities, and possibly contributed to the activation of geologic faults. For example, since 1943 about 8 km^2 (3 sq mi), an area containing over 400 homes, has been submerged due to subsidence in excess of 2.4 m (8 ft) along the tidal reaches of Galveston Bay.

The concluding paper, by J. G. Newton, describes the kind of subsidence that is dramatic but local in nature, such as that occurring in carbonate (karstic) terrane in the southeastern U.S. The author emphasizes problems created by "induced" sinkholes—those related to man's activities, such as decline in the water table due to excessive ground-water withdrawal. The water table decline results in a loss of buoyant support, increase in the velocity of movement of water, water-level fluctuations at the base of the unconsolidated deposits, and induced recharge. The author noted that the sinkholes occur where cavities develop in residual or other unconsolidated deposits overlying openings in carbonate rocks and that the downward migration of the deposits into the underlying openings cause sudden collapse of the sediments and formation of sinkholes from 1 m (3.3 ft) to 100 m (330 ft) in diameter and 0.3 m (1 ft) to 30 m (99 ft) in depth. Around 4000 sinkholes in Alabama have resulted in damage of many millions of dollars to highways, railroads, pipelines, buildings, and other construction.

These five papers present only a small amount of information on the problems that can be created by land subsidence. They emphasize only a few of the problem areas found in the United States; the subsidence problems are as great or greater in other parts of the world, e.g., Mexico, Italy, and Japan. It is hoped that this small group will at least make many more people aware of the multitude of legal, socio-economic, and environmental problems that can be created by land subsidence.

SUBSIDENCE IN UNITED STATES DUE TO GROUND-WATER WITHDRAWAL[a]

By Joseph F. Poland[1]

INTRODUCTION

This paper is a summary of land subsidence due to ground-water withdrawal in the United States. The chief source of data is a casebook being prepared for the United Nations Educational, Scientific, and Cultural Organization (UNESCO) by an international working group, of which the writer of this paper is the chairman. Other sources of data are: (1) The Questionnaire on Land-Subsidence Occurrence, Research, and Remedial Work that was distributed worldwide in 1975–1978 by Ivan Johnson, in behalf of International Association of Hydrological Sciences (IAHS); and (2) the Proceedings of the 2nd International Symposium on Land Subsidence held at Anaheim, Calif., in December 1976.

The goal of the casebook is to produce a guide to assist engineers and geologists in the study, evaluation, and control of land subsidence, especially in developing countries.

The casebook contains a manual covering the occurrence, measurement, mechanics, prediction, and control of subsidence, and 16 invited case histories of land subsidence due to ground-water withdrawal. The case histories cover a wide range of conditions and magnitude of subsidence; four are for areas in United States. The casebook also includes a tabulation of 41 subsidence areas, of which 17 are in the United States (exclusive of sinkhole areas). Fig. 1 shows the location of the 17 areas on a map of conterminous United States. Subsidence of the land surface in the 17 areas ranges from 0.3 m at Savannah, Ga., to 9.0 m on the west side of the San Joaquin Valley (Los Banos-Kettleman City area) in California (Fig. 2). Subsidence exceeding 1 m occurs in four states: Texas, Arizona, Nevada, and California. The areal extent ranges from 10 km^2 in San Jacinto Valley, Calif., to 13,500 km^2 in the San Joaquin Valley. California is the state ranking number one for the dubious honor of having the largest

[a]Presented at the October 22–26, 1979, ASCE Annual Convention and Exposition, held at Atlanta, Ga.

[1]Research Hydro., U.S. Dept of the Interior, Geological Survey, Water Resources Div., Room W-2528, Federal Building, 2800 Cottage Way, Sacramento, Calif., 95825.

Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0115/$01.00.

area of subsidence—about 16,000 km^2. Close behind is Texas with 12,000 km^2 and Arizona is third with 2,700 km^2.

Principal problems caused by the subsidence are: (1) Differential changes in elevation and gradient of stream channels, drains, and watertransport structures;

FIG. 1.—Areas of Land Subsidence from Ground-Water Withdrawal, USA

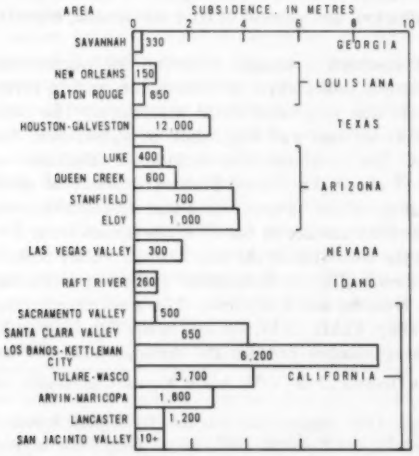

FIG. 2.—Magnitude of Land Subsidence from Ground-Water Withdrawal, USA (Number in Column Represents Area in Square Kilometers)

(2) failure of water-well casings due to compressive stresses generated by compaction of aquifer systems; and (3) tidal encroachment in lowland coastal areas.

Geological Environments of Occurrences

Subsidence due to ground-water withdrawal develops principally under two contrasting environments and mechanics. One environment is that of carbonate rocks overlain by unconsolidated deposits, or old sinkholes filled with unconsolidated deposits, that receive buoyant support from the ground-water body. When the water table is lowered, buoyant support is lost and the hydraulic gradient increased; the unconsolidated material may move downward into openings in the underlying carbonate rocks, sometimes causing catastrophic collapses of the roof. In Alabama, an estimated 4,000 man-induced sinkholes have formed since 1900 in contrast to less than 50 natural collapses (16). In the United States the occurrence of manmade sinkholes is common in carbonate terrane from Florida to Pennsylvania, numbering many thousands. The individual sinkhole area is small, however, usually within the range of 10,000 m^2. This writer will say no more about subsidence in carbonate terrane because that is the topic of a companion paper by John Newton.

The other and by far the most extensive occurrence is that of young unconsolidated or semiconsolidated sediments of high porosity laid down in alluvial, lacustrine, or shallow marine environments. All areas are underlain by semiconfined or confined aquifer systems containing aquifers of sand or gravel, or both, of high permeability and low compressibility interbedded with clayey aquitards of low vertical permeability and high compressibility under virgin stresses. Most of the compacting deposits were normally loaded, or approximately so, before man applied stresses exceeding preconsolidation stress (maximum past stress). These aquifer systems compact in response to increased effective stress caused by artesian-head decline in the coarse-grained aquifers and time-dependent pore-pressure reduction in the fine-grained compressible aquitards, causing land-surface subsidence. Note that from Savannah, Ga., to Houston, Tex. (Fig. 1), the ground-water withdrawal and subsidence are occurring in coastal-plain and shallow marine deposits, whereas from Arizona to northern California, the withdrawal and subsidence are occurring in valley fill (Ref. 13, Fig. 100), chiefly alluvial-fan and flood-plain deposits.

Stresses that Cause Subsidence

The idealized pressure diagram of Fig. 3 can be utilized to show the stresses that cause subsidence.

In 1925, Terzaghi introduced the theory of effective stress:

$$p' = p - u_w \quad \ldots\ldots\ldots\

Therefore, the increase in effective stress in the confined aquifers is equal to the decrease in fluid pressure. The compaction of these aquifers is immediate and is chiefly recoverable if fluid pressure is restored, but the amount is small.

The decrease in head in the aquifers creates an hydraulic gradient from the confining clays and the aquitards to the aquifers. However, these fine-grained beds have low hydraulic conductivity and high compressibility. Therefore, the vertical escape of water and the consequent decrease in pore pressure and increase in effective stress are slow and time-dependent, but the ultimate unit compaction is large and chiefly permanent. It is the time-dependent nature of the stress increase in these fine-grained beds that complicates the problem of predicting subsidence. In these aquitards the stress increase applied by the head decline in the confined aquifers becomes effective only as rapidly as pore pressures decay toward equilibrium with the pressures in adjacent aquifers. (See dashed pore-pressure lines of Fig. 3, where u_t represents the excess pore pressure

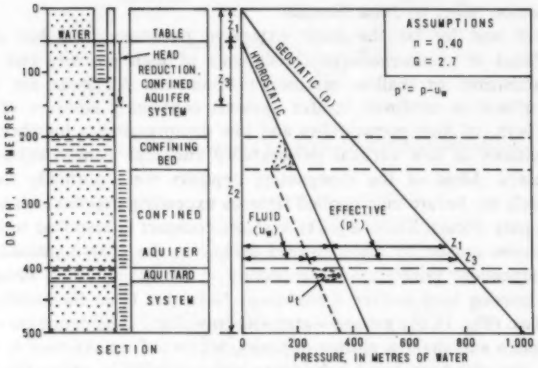

FIG. 3.—Pressure Diagram for Unconfined Aquifer and Confined Aquifer System; Head Reduction in Confined System only

at time t.) Attainment of pore-pressure equilibrium (dotted lines) may take months or years; the time required to reach equilibrium varies directly as the specific storage and the square of the draining thickness and inversely as the vertical hydraulic conductivity of the aquitard.

The stress relations of Fig. 3 show the principle of effective stress but not the hydrodynamic cause of compaction. Actually, the downward hydraulic gradient developed across the confining bed by the head decline induces downward movement of water through the pores and exerts a viscous drag on the clay particles. This downward force due to viscous drag is exerted from particle to particle throughout the aquifer system. The force transferred to the particles at any depth is measured by the head loss to that depth. The stress so exerted on the particles in the direction of flow is a seepage stress. Lofgren (1968) has pointed out that in treating complex aquifer systems, it is quantitatively convenient to compute effective stresses and stress changes in terms of gravitational and seepage stresses, which are algebraically additive. Gravitational stress

is caused by the effective weight of overlying deposits, transmitted downward through the grain-to-grain contacts. Any change in the position of the water table changes the gravitational stress on underlying deposits.

Seven years ago the writer prepared a paper reviewing subsidence in the United States caused by ground-water withdrawal (18). The remainder of the present paper is chiefly an update of subsidence developments.

Subsidence Areas

Savannah, Ga.—The latest information available on the subsidence of the Savannah area, Ga., has been reported by Davis, Counts, and Holdahl (5).

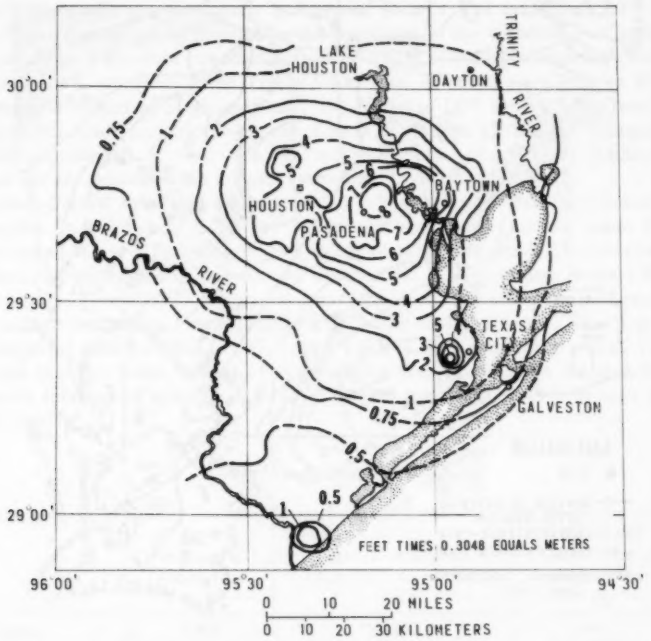

FIG. 4.—Land Subsidence, Houston-Galveston Region, Tex., 1943–1978, in feet (Modified from Gabrysch, R. K., 1980, *U.S. Geological Survey Open-File Report, 80-338,* Fig. 2)

Subsidence to 1975 was 0.3 m; it has not been sufficient to be recognized as a serious engineering problem to date.

Louisiana.—Information available on subsidence due to ground-water withdrawal in Baton Rouge and New Orleans, La., as of 1972 was summarized briefly by the writer (18). Subsidence in Baton Rouge was 0.4 m as of 1969 and in New Orleans was 0.55 as of 1964. Total subsidence in the Baton Rouge

industrial area 1935–1976 was 0.51 m, of which 0.38 m was attributed to ground-water withdrawal and about 0.13 m due to natural regional subsidence, assumed to be at a rate of 3 mm/yr (23). R. G. Kazmann reported (written communication, December 1975) that maximum subsidence in New Orleans had increased to 0.8 m by 1975. So far as is known, no remedial steps have been taken to date.

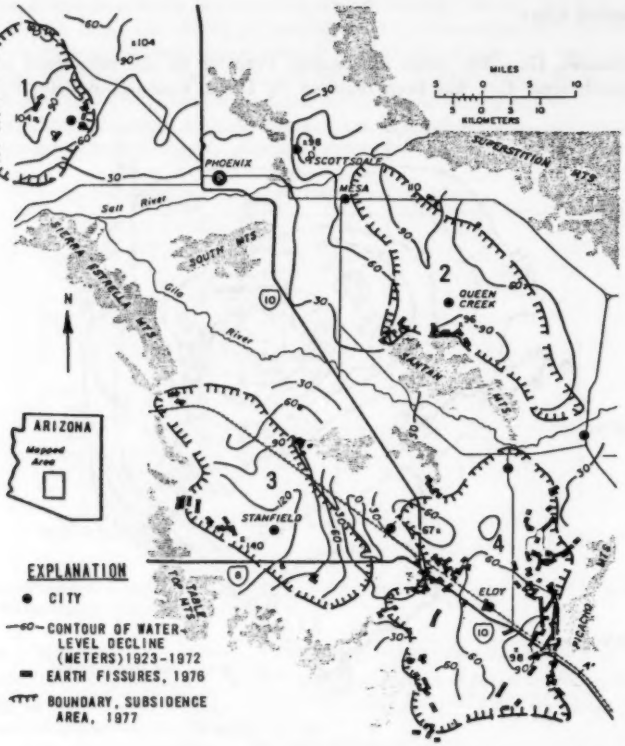

FIG. 5.—Water-Level Declines, Earth Fissures, and Subsidence Areas in South-Central Arizona, Modified from Winikka and Wold (1977, Fig. 1); Subsidence Areas Numbered as follows: 1, Luke; 2, Queen Creek; 3, Stanfield; 4, Eloy; Areas are as Shown on IAHS Questionnaires Prepared by C. C. Winikka, January 1978

Houston-Galveston Area, Tex.—The Houston-Galveston area had experienced maximum subsidence of 2.75 m (9 ft) by 1975 (Gabrysch, R. K., written communication, December 1975). The bench-mark net in the subsiding area was resurveyed in 1978–1979. Fig. 4 shows the subsidence from 1943 to 1978. As much as 2.6 m (8.5 ft) of subsidence occurred in the Pasadena area in

the 350-yr period. From 1890 to 1954, all the water supply for the Houston-Galveston area was ground water. In 1954, surface water first became available from Lake Houston (Fig. 4). By 1972, pumping of ground water for municipal supply (46%), industrial use (33%), and irrigation (21%) totaled 2 hm^3/day (525,000,000 gal/day). In 1975, pumping of ground water for all uses was 1.9 hm^3/day.

In the Pasadena and Baytown areas, the long-term increasing withdrawal of ground water caused maximum water-level declines from 1943 to 1973 of 61 m in wells completed in the Chicot (shallower) aquifer and 99 m in wells completed in the Evangeline aquifer (6). Both are confined aquifer systems. This long-term drawdown of water levels (increase in effective stress in the aquifers) has been the cause of the land-surface subsidence.

The Harris-Galveston Coastal Subsidence District was created in 1975 by the Texas Legislature to provide for the regulation of the withdrawal of ground water within the district boundaries for the purpose of ending subsidence. Water from Lake Livingston on the Trinity River (Fig. 4) became available in 1976. Substantial reduction of ground-water withdrawal in 1977 and 1978 has resulted in artesian-head recoveries in excess of 30 m in both the Chicot and Evangeline aquifer systems (R. K. Gabrysch, oral communication, August 1979). Additional rises are anticipated as the imports from Lake Livingston increase.

South-Central Arizona.—Ground water has been mined in Arizona for several decades. In 1975 about 6,200 hm^3 of ground water was pumped; about 95% for irrigation use (15). Probably two thirds of this was mined. The overdraft, which has persisted for at least two decades has lowered water levels 100 m or more in several heavily pumped areas. This water-level decline has caused subsidence exceeding 1 m in four areas of south-central Arizona. Fig. 5 shows water-level declines 1923 to 1972, earth fissures, and subsidence areas. Table 1 lists number, name, and size of subsidence areas, as well as the maximum known subsidence, and the water-level decline from 1923 to 1977 in each area (8).

TABLE 1.—Subsidence Areas

Name (1)	Area, in square kilometers (2)	Water-level decline, 1923–1977, in meters (3)	Maximum subsidence, in meters (year) (4)
Luke	400	90–100	>1 (1967)
Queen Creek	600	90–120	1.5 (1976)
Stanfield	700	90–140	3.6 (1977)
Eloy	1,000	60–90	3.8 (1977)

Additional information on the subsidence and fissuring is available in several maps and reports published in the past 5 yr, including but not limited to Refs.

7, 8, 22 and 24. Water levels are continuing to decline, land subsidence is increasing in these areas of increasing stress, and new earth fissures are developing, all as a result of the severe continuing overdraft on ground-water resources. The Central Arizona Project of the U.S. Bureau of Reclamation, when completed, will alleviate subsidence in some basins but the additional water supply of 1,480 hm^3 is less than one half of the estimated overdraft.

Las Vegas Valley, Nevada.—For a summary of data relating to land subsidence in Las Vegas Valley as of 1970, the reader is referred to a report by Anthony Mindling (14). He reported maximum subsidence of 1.2 m from 1935 to 1963, based on leveling of the National Geodetic Survey.

A ground-water pumpage inventory made by the State of Nevada Division of Water Resources indicated withdrawal of 85 hm^3 in Las Vegas Valley during

FIG. 6.—Area of Land Subsidence in California due to Ground-Water Withdrawal

1978. The first unit of the Southern Nevada Water Project, to bring Colorado River water to the Las Vegas area, was completed in 1971 with a design capacity of 163 hm^3/yr. In 1978 about 114 hm^3 of Colorado River water was used in Las Vegas Valley, of which roughly 17 hm^3 of Colorado River water was used in the vicinity of Henderson in the southeast part of the valley. The second unit of the Southern Nevada Water Project is scheduled for completion and initial water delivery late in 1981. The design capacity is 206 hm^3/yr. After completion of the second unit, the total design capacity of the project to bring Colorado River water to Las Vegas Valley will be about 370 hm^3/yr. The total of the long-term permits to pump ground water is about 62 hm^3/yr. After the shorter term permits expire, it is anticipated that water available from the

Colorado River will be adequate to permit holding ground-water withdrawal to the 62 hm^3/yr and slow down or stop the artesian-head decline. This should alleviate the problem of subsidence and fissuring in the Las Vegas area in the 1980s, except in local areas of intensive ground-water extraction.

California.—The two principal areas of land subsidence due to ground-water withdrawal in California are the San Joaquin Valley and the Santa Clara Valley (Fig. 6). Case histories of these two areas have been included in the UNESCO casebook. Two series of Geological Survey Professional Papers have been published. Professional Papers in the 437 number series represent products of

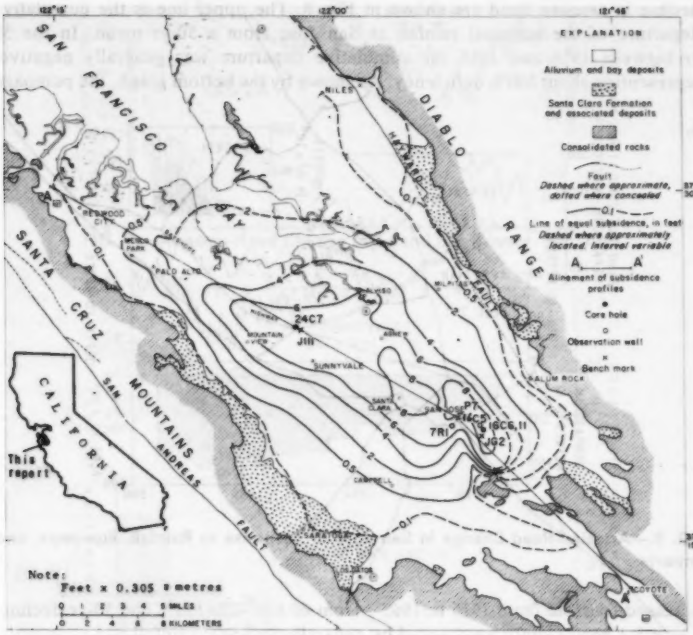

FIG. 7.—Land Subsidence from 1934 to 1967, Santa Clara Valley, Calif.; Compiled from Leveling of National Geodetic Survey in 1934 and 1967

land-subsidence studies in cooperation with the California Department of Water Resources; those in the 497 number series are products of the Federal program on mechanics of aquifer systems. Three other areas of smaller subsidence, not considered in this paper, are the Sacramento Valley (0.7 m), the Lancaster area (1 m, 1955–1976), and the San Jacinto Valley (1 m, 1950–1974).

Santa Clara Valley.—Land Subsidence in the Santa Clara Valley was first noted in 1933. By 1969 the central part of the city of San Jose had subsided about 4 m. Fig. 7 shows the extent and magnitude of subsidence in the 33-yr

period 1934–1967. It ranged from 3 ft (1 m) or more beneath San Francisco Bay to 8 ft (2.5 m) in San Jose.

In the central two-thirds of the valley below a depth of about 60 m, ground water is confined. The extent of the subsidence defines in a general way the extent of the confined system. Specifically, in Fig. 7, the boundary of the confined system is between the 0.5-ft and the 2-ft lines, about along the 1-ft subsidence line (interpolated).

The historical development of land subsidence was in response to a major decline in artesian head (19). From 1916 to 1966 the artesian head in index well 7R1 in San Jose declined 58 m. The two factors that caused this major decline in artesian head are shown in Fig. 8. The upper line is the cumulative departure of the seasonal rainfall at San Jose from a 50-yr mean. In the 50 yr between 1916 and 1966 the cumulative departure was generally negative, representing about 300% deficiency. As shown by the bottom graph, the pumpage

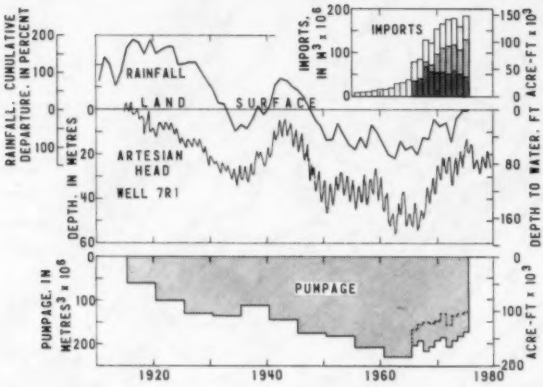

FIG. 8.—Artesian-Head Change in San Jose in Response to Rainfall, Pumpage, and Imports

increased fourfold from 1915 to 1965—from 60 hm^3–228 hm^3. The 50-yr decline in artesian head plainly was caused by generally deficient rainfall and constantly increasing pumping draft.

The subsidence record for bench mark P7 in San Jose is shown in Fig. 9, together with the artesian head in nearby index well 7R1, taken from Fig. 8. The fluctuation of artesian head reflects the change in stress on the confined aquifer system: the subsidence is the resulting strain. The cause and effect relationship is clear. Between 1912 and 1967 bench mark P7 subsided 3.86 m.

The recovery of artesian head from 1967 to 1975 was dramatic; the head in two index wells recovered about 30 m in the 8 yr. As documented in Fig. 8, recovery of artesian head was due to a fivefold increase in surface-water imports from 1965 to 1975, favorable local water supply from rainfall, and decreased pumpage of ground water. Also, the elastic (recoverable) storage coefficient was functional in both aquifers and aquitards during this recovery

stage, whereas during the long-term drawdown about 620 hm^3 of water was derived from aquitard compaction in response to the much larger virgin storage coefficient.

Although the bench-mark net in the subsiding area has not been resurveyed since 1967, two vertical extensometers installed by the Geological Survey to a depth of 305 m have furnished continuous measurements of compaction of the confined-aquifer system since 1960 (approximately equal to subsidence).

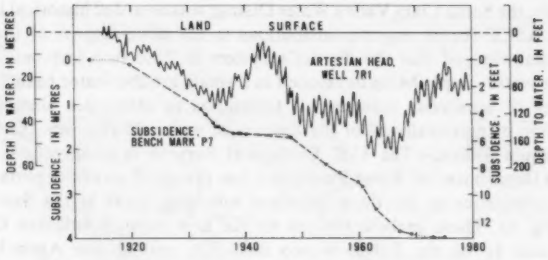

FIG. 9.—Artesian-Head Change and Land Subsidence, San Jose

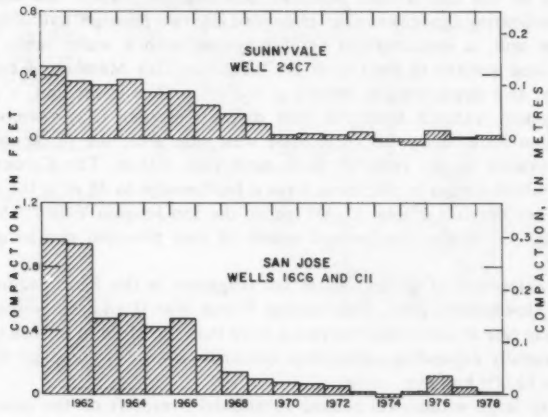

FIG. 10.—Measured Annual Compaction to 305 m (1,000 ft) Depth

Fig. 10 shows the annual compaction at the two sites from 1961 through 1978. In San Jose (wells 16C6 and C11) the rate decreased from 30 cm in 1961 to 0.3 cm in 1973. Net expansion (land-surface rebound) of 0.6 cm occurred in 1974.

The successful management of a highly variable water supply to achieve a balance with an ever-increasing demand for water in Santa Clara County (not shown on map) has been remarkable for several reasons. First, maximum

development of local water supplies and importation of water from two sources have momentarily brought supply and demand into balance. Secondly, by building up the ground-water storage in the recharge area, and the artesian head in the confined system, land subsidence was stopped, at least temporarily, by 1975. Thirdly, all this has been accomplished by bond issues, revenue from taxes, and water charges, thus avoiding a drawn-out expensive legal adjudication of the ground-water supply, such as occurred in southern California in the Raymond Basin (17).

Recently, the Santa Clara Valley Water District was awarded historical landmark status by ASCE for its major contributions to the development of the region. It was acknowledged that the district's system is "the first and only instance of a major water supply being developed in a single ground-water basin involving the control of numerous independent tributaries to effectuate almost optimal conservation of practically all of the sources of water flowing into the basin."

San Joaquin Valley.—The U.S. Geological Survey, in cooperation with the California Department of Water Resources, has prepared several reports describing land subsidence in the three principal subsiding areas in the San Joaquin Valley (Fig. 6). These include reports on the Los Banos-Kettlemen City area (1, 2, 3, and 4), on the Tulare-Wasco area (12), and on the Arvin-Maricopa area (10). A report by the writers, Lofgren, Ireland, and Pugh (20) is a summary of land subsidence in the valley as of 1972.

In most of the central and southern San Joaquin Valley, the continental freshwater-bearing deposits can be subdivided into two principal hydrologic units. The upper unit, a semiconfined aquifer system with a water table, extends from the land surface to the top of the Corcoran Clay Member of the Tulare Formation, at a depth ranging from 0 m to 275 m. The lower unit, a confined aquifer system, extends from the base of the Corcoran Clay Member down to the saline water body. In the central west side area, the thickness of this confined system ranges from 60 m to more than 600 m. The Corcoran Clay Member, which ranges in thickness from a featheredge to 48 m is the principal confining bed beneath at least 13,000 km^2 of the San Joaquin Valley. The dotted line in Fig. 11 defines the general extent of this principal confining bed in the valley.

Yearly extraction of ground water for irrigation in the San Joaquin Valley increased slowly until 1940. Then during World War II and the following two decades, the rate of extraction increased more than threefold to furnish irrigation water to rapidly expanding agricultural demands. By 1966, pumpage of ground water was 12,000 hm^3/yr.

This very large withdrawal caused substantial overdraft on the central west side and in much of the southern part of the valley, essentially within the shaded area of Fig. 11. The withdrawal in these overdraft areas in the 1950s and early 1960s was at least 5,000 hm^3/yr. As a result, the head (potentiometric surface) in the confined aquifer system between Los Banos and Wasco was drawn down 60 m–180 m. South of Bakersfield the head decline was more than 100 m.

Importation of surface water to these areas of serious overdraft began in 1950 when water from the San Joaquin River was brought south through the Friant-Kern Canal, which extends to the Kern River (Fig. 11). About 80% of the average annual deliveries of 1,250 hm^3 of water from this canal is sold

to irrigation districts south of the Kaweah River, mostly in the Tulare-Wasco subsidence area.

Large surface-water imports from the northern part of the state to overdrawn areas on the west side and south end of the valley beginning in 1967 are being supplied through the California Aqueduct (Fig. 11). The joint-use segment of the aqueduct between Los Banos and Kettleman City serves the San Luis project area of the U.S. Bureau of Reclamation and transports state-owned water south

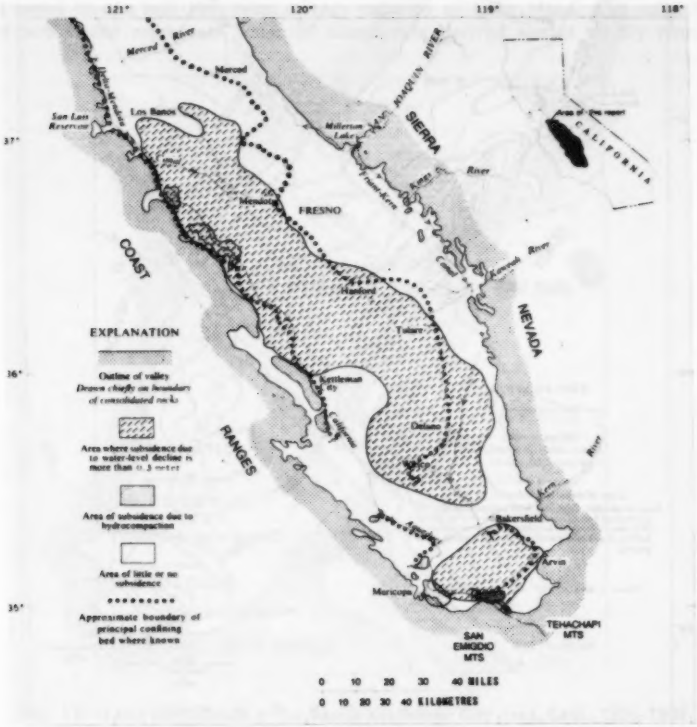

FIG. 11.—Pertinent Geographic Features of Central and Southern San Joaquin Valley and Areas Affected by Subsidence

of Kettleman City. By 1973 the California Aqueduct delivered 1,920 hm^3 to the central and southern parts of the San Joaquin Valley.

As a result of these large surface-water imports, the rate of ground-water withdrawal decreased sharply and the decline of artesian head was reversed in most of the areas of overdraft. By the early 1970s many hundreds of irrigation wells were unused, artesian heads were recovering at a rapid rate, and rates of subsidence were sharply reduced.

Subsidence in the San Joaquin Valley is of three types. In descending order of importance these are: (1) Subsidence due to the compaction of aquifer systems caused by the excessive withdrawal of ground water; (2) subsidence due to the compaction of moisture-deficient deposits when water is first applied—a process known as hydrocompaction; and (3) local subsidence caused by the extraction of fluids from several oil fields.

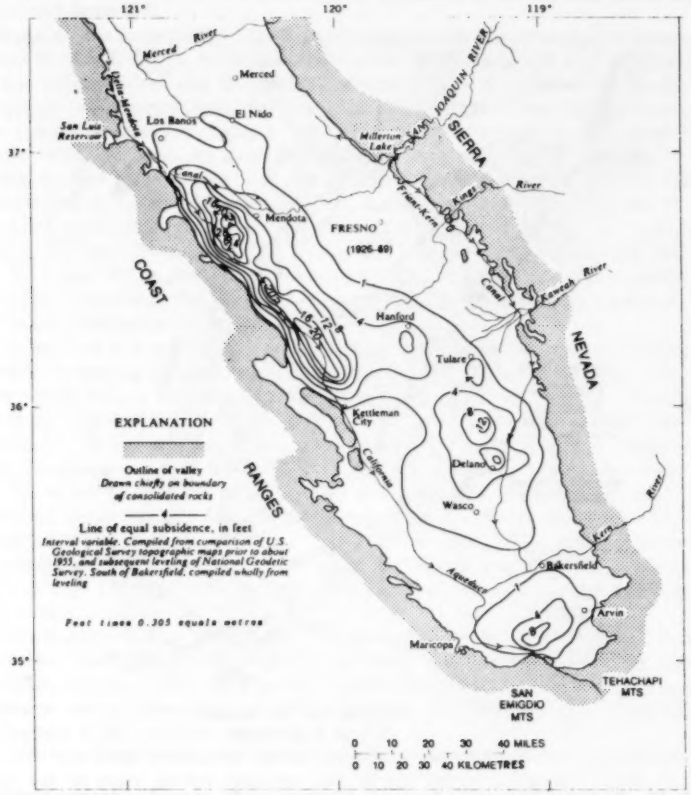

FIG. 12.—Land Subsidence in San Joaquin Valley, California, 1926–1970

Subsidence due to the compaction of aquifer systems in response to excessive decline of water levels had affected about 13,500 km^2 of the San Joaquin Valley by 1970. Fig. 12 depicts the distribution and magnitude of subsidence exceeding 1 ft (0.3 m) that had occurred by 1970—affecting an area of 11,100 km^2. Three centers of subsidence stand out on this map. The most prominent is the long narrow trough west of Fresno that extends 140 km from Los Banos to Kettleman

City (referred to subsequently as the west side area). Maximum subsidence in this area to 1972 was 29 ft (8.8 m), 16 km west of Mendota. The second center, between Tulare and Wasco, is defined by two closed 12-ft (3.7-m) lines of equal subsidence, 32 km and 48 km south of Tulare, respectively. The third center, south of Bakersfield, has subsided a maximum of 8 ft (2.4 m), mostly since World War II.

The cumulative volume of subsidence in the San Joaquin Valley soared to 12,350 hm^3 by 1960, and reached 19,250 hm^3 by 1970. This very large volume is equal to one half the initial storage capacity of Lake Mead. The volume of subsidence represents water of compaction derived almost wholly from

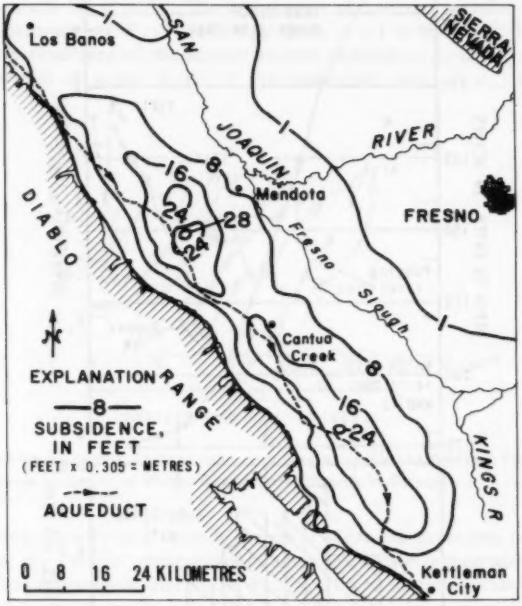

FIG. 13.—Land Subsidence in Los Banos-Kettleman City Area, Calif., 1926–1969

compaction of the fine-grained highly compressible clayey interbeds (aquitards), in response to the increase in effective stress as artesian head in the confined system declined.

The west side area has experienced the most severe subsidence (Fig. 13); therefore, several illustrations will be presented to show the relation between water-level change (stress change) and compaction or subsidence, or both, in that area.

Fig. 14 shows the relation of subsidence to artesian-head change since 1943 at a site 16 km southwest of Mendota. Bench mark S661, located within the 28-ft (8.5-km) line of equal subsidence in Fig. 13, subsided 8 m from 1943

to 1969, in response to a water-level decline of more than 100 m as measured in nearby wells. The rate of subsidence at this site reached a maximum of 0.54 m/yr between 1953 and 1955 but decreased to 0.04 m/yr between 1972 and 1975, due chiefly to substantial recovery of artesian head. Static water level began to recover in 1969 and by 1977 had risen 73 m above the 1968 summer low level because of the great reduction in ground-water pumpage in response to the availability of large imports of surface water through the California Aqueduct.

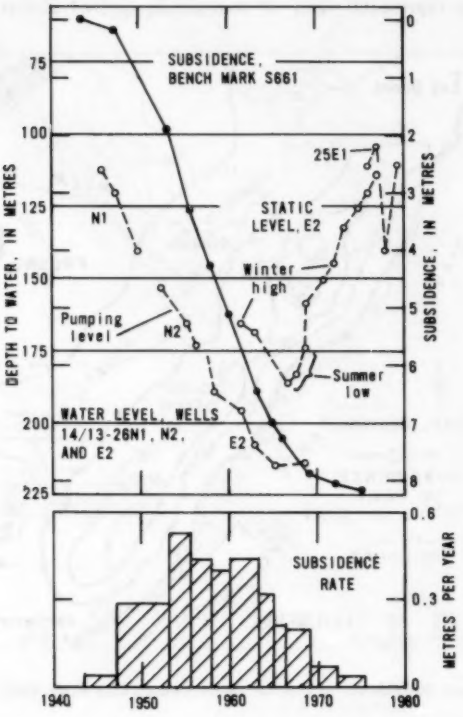

FIG. 14.—Subsidence and Artesian-Head Change 16 km Southwest of Mendota

Field measurements of compaction or expansion of sediments and the correlative change in fluid pressure(s) can be utilized to construct stress-strain curves and to derive storage and compressibility parameters. One example (Fig. 15) is for a well 176 m deep on the west side of the valley. Depth to water is plotted increasing upward (increasing stress). Change in depth to water represents change in effective stress in the aquifers in the confined aquifer system (upper zone) that is 106 m thick. Along the abscissa the lower scale is the measured

compaction and the upper scale is the strain (measured compaction/compacting thickness). The yearly fluctuation of water level caused by the seasonal irrigation demand and the permanent compaction that occurs each summer during the heavy pumping season when the depth to water is greatest produce a series of stress-strain loops. The lower parts of the descending segments of the annual loops for the three winters 1967–1968 to 1969–1970 are approximately parallel straight lines, indicating that the response is essentially elastic in both aquifers and aquitards when the depth to water is less than about 55 m. The heavy dashed line in the 1968 loop represents the average slope of the segments in the elastic range of stress. The reciprocal of the slope of the line is the component of the storage coefficient due to deformation of the aquifer-system skeleton, S_{ke}, and equals 1.2×10^{-3}. The component of average specific storage due to elastic deformation, S_{ske}, equals $S_{ke}/106\ \text{m} = 1.1 \times 10^{-5}\ \text{m}^{-1}$. The average elastic compressibility of the aquifer system skeleton, α_{ke}, is S_{ske}/γ_w; if γ_w, the unit weight of water, equals 1, the numerical values of α_{ke} and S_{ske} are identical.

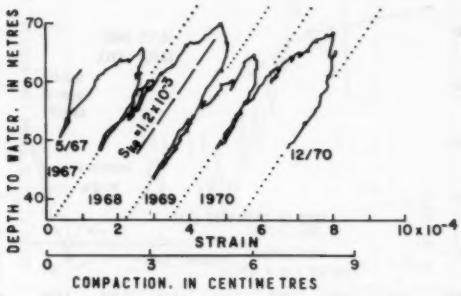

FIG. 15.—Stress Change, Compaction, and Strain for Well in Western Fresno County, Calif. (Change in Depth to Water Represents Change in Stress)

For increase in effective stress in the range of loading exceeding preconsolidation stress, the "virgin" compaction of aquitards is chiefly inelastic—nonrecoverable upon decrease in stress. At Pixley, 27 km south of Tulare (Fig. 12), compaction and change in stress for a confined aquifer system 108 m–231 m below land surface has been measured since 1958. Riley (21) showed from a stress-strain plot that the mean virgin compressibility of the aquitards (aggregate thickness 75 m) in this confined aquifer segment 123 m thick was 7.5×10^{-4} m^{-1} and the mean elastic compressibility of the aquifer system was 9.3×10^{-6} m^{-1}. Thus, for the aquifer system segment 123-m thick at this site the mean virgin compressibility of the aquitards is about 80 times as large as the mean elastic compressibility of the confined system.

Fig. 16 shows a generalized plot of water level for the confined aquifer system 32 km south of Mendota (Fig. 13) from 1905 to 1964 and the seasonal high and low in observation well 16/15-34N4 for 1961–1977. This well taps the confined system. The regional water level declined about 120 m from 1905 to 1960 and the rate of decline accelerated as the ground-water withdrawal increased.

Surface-water imports to the west side area began in 1968. As the imports increased, ground-water pumpage decreased and water levels recovered sharply. From 1968 to 1976 the water level at well 34N4 rose 82 m. Then, during 1977, the second of two severe drought years, the imports dropped to about 20% of 1976 imports and pumping draft from both old and newly drilled wells soared to about three times the 1976 rate. As a result of water level fell 50 m in the 8 months to August 1977.

The changing stress as indicated by the hydrograph of well 34N4 and the resulting strain at this site as measured by an extensometer in well 34N1 since 1959 are clearly displayed in Fig. 17. Well 16/15-34N1, 610 m deep, is equipped with an anchored-cable extensometer. The measured compaction is plotted as an annual bar graph for comparison with the fluctuations of the water level in well N4. Note that the water level in well N4 began a sharp rise late in 1968 as surface-water imports began. In response to the sharp recovery of

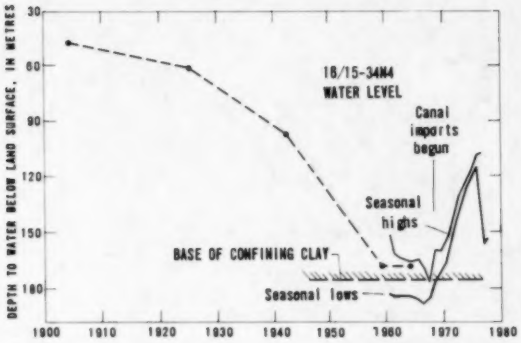

FIG. 16.—Long-Term Trend of Water Levels near Cantua Creek, and Seasonal High and Low Levels in Observation Well 16/15-34N4 since 1960 (Modified from Ref. 11, Fig. 8)

water level, compaction decreased rapidly after 1968 but did not cease until 1975. During this period of rising water levels in the coarse-grained aquifers, nonrecoverable virgin compaction continued through 1974 in the central parts of the thicker aquitards, exceeding the continuing small elastic expansion of the preconsolidated aquifers and the thinner aquitards. The water level in well N4 reached a seasonal high of 107 m below land surface in November 1976. We know from the detailed records that early in February 1977, when water level was 112 m below land surface (only 5 m below the seasonal high), virgin compaction resumed in well N1. By March 30, 1977, when water level was 15 m below the seasonal high, the maximum compaction rate of the season was attained. The early February water level 112 m below land surface clearly defined the preconsolidation stress in the central segments of the thickest or least permeable aquitards, or both. As the drawdown increased, more and more of the slow draining compressible beds began to contribute water of compaction.

By the end of the year, about 12 cm of renewed nonrecoverable compaction had occurred.

During the first period of water-level decline (1905–1968 in Fig. 16), water of compaction represented about one third of the total water pumped from west side wells (Ref. 20, Fig. 19). By 1968, many of the aquitards were preconsolidated nearly to the 1968 stress level. Early in the second period of water-level decline (in 1977), the response of the preconsolidated sediments was chiefly elastic and the contribution of water of compaction was much less than one third of the total pumpage. Thus, the water level fell very rapidly.

The storage capacity represented by compaction of a compressible aquifer system has two components (neglecting the compressibility of water). One

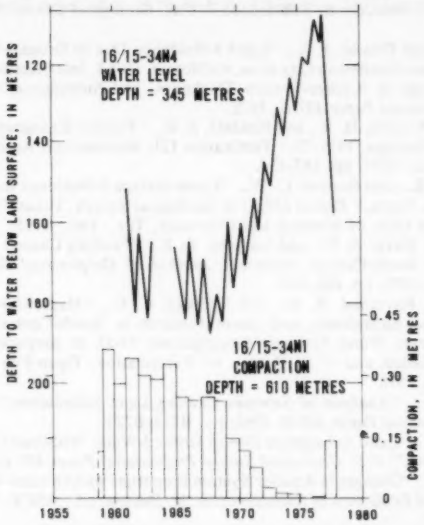

FIG. 17.—Seasonal Fluctuations of Water Level in Well 16/15-34N4 and Measured Compaction in Observation Well 16/15-34N1 near Cantua Creek (Modified from Ref. 11, Fig. 10)

component is the volume released through the elastic (recoverable) deformation of the entire system that occurs at stresses less than preconsolidation stress. The second component is the volume released by the nonrecoverable permanent compaction of the aquitards and confining beds that occurs at stresses exceeding preconsolidation stress. The *usable* storage capacity of an aquifer system commonly is defined as the volume *than can be taken from* or *recharged to* the system. The pore volume eliminated by the permanent compaction of the aquitards cannot be recharged. Therefore, the *usable* storage capacity, as previously defined, includes only the first component—the volume released through the elastic deformation of the aquifer system. For highly compressible

aquifer systems, like those in the San Joaquin Valley, the volume released through elastic deformation per unit change of head is very small compared to the volume released through inelastic deformation under virgin stressing.

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LEGAL ASPECTS OF SUBSIDENCE DUE TO WELL PUMPING[a]

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INTRODUCTION

The Santa Clara and San Joaquin Valleys of California have suffered substantial subsidence from overpumping of ground water, with drops of 8.5 m (28 ft) in one region of the San Joaquin Valley. Ground-water overdraft and related subsidence have created critical conditions in these valleys and elsewhere. Damages include destruction of wells and other utilities, reversal of drainage gradients, and inundation of lands. In the San Joaquin Valley, the locations of streams are beginning to be affected by subsidence. California's ground water is generally available to any pumper regardless of the effect of extraction on neighboring ground water pumpers or on subsidence of the land.

The mechanism of subsidence of land due to pumpage of ground water is not always the same; however, there is a general pattern. When underground water is pumped, there is a rapid decrease in hydraulic pressure which partially supports the weight of the overburden. With the reduction in pressure, an additional load is placed upon the skeleton of the aquifer. If the aquifer contains layers of clay and porous materials, the pressure differences between the sands and clays may cause water to move from the clays to the sands. Subsidence follows from the compaction of clays, and is permanent because the clays are inelastic. Thus, permanent sinking of the ground occurs.

This paper, in the first section, explores the legal roots of the nation's *laissez faire* attitude toward ground water pumpage and subsidence. In the second section, it covers some of the important legal precedents which indicate a shift in policy toward holding pumpers liable for subsidence. The final section examines the extent of subsidence problems in the nation today, and innovative programs to manage ground water basins and prevent subsidence.

[a]Presented at the October 22–26, 1979, ASCE Annual Convention and Exposition, held at Atlanta, Ga.

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0137/$01.00.

The first instances of subsidence were caused by the removal of the minerals beneath the surface land. Early cases dealt with coal mining where, through negligence, the mining company removed the pillars of support which held the surface land in place. It was relatively late in the development of the English common law when cases arose involving subsidence caused by removal of ground water. Generally, these cases held that the person who pumped the ground water was not liable for the damage to his neighbor's property under the principle of *damnum absque injuria*; i.e., there is no remedy at law for the injury.

Subsidence cases fall into two groups—those subsidences caused by the removal of minerals and those caused by the removal of liquids. Frequently, courts have applied the same principles of law to subsidences caused by the removal of liquids as to those caused by removal of solids. In reality, the situations are quite different. When coal or other solids are removed, it is always possible to leave pillars to support so that the ground does not subside. Such would not be the case with the mining of liquids such as oil, asphalt, or water.

Subsidence caused by the removal of solids is usually due to negligence. The contractor who removes the solids simply does not bother to provide the necessary support to keep the surface estate intact. Subsidence caused by the removal of ground water generally has not been regarded as related to negligence. It is useful to first view cases which decide liability for the removal of solid minerals and to compare them with ground-water cases.

Rule of Law for Subsidence Caused by Removal of Solid Minerals.—The English common law rule is that subsidence caused by the removal of minerals creates an absolute liability without fault for subsidence damage to land in its natural state. The famous American case of *Pennsylvania Versus Mahon* took the opposite view. It held the surface estate did not have the right to support from the subsurface. However, subsequently the English rule was adopted.

The issue which generally arises is how much of the surface estate was deeded away with the rights to mine coal. A question also occurs as to whether the royalty interest owner should have a right to recover when he owns part of the property impacted by the subsidence. It is argued that he should also be liable for the subsidence since he receives a monetary benefit from the mining activity beneath the surface estate. Cases involving solid minerals have consistently limited liability to the person who removed the support, except: (1) Where such person was a lessee and the lessor had actual control over the operations; or (2) where the lessor, who was receiving royalties from the minerals, knowingly permitted the lessee to remove subjacent support in violation of the lease.

Subsidence Caused by Withdrawal of Ground Water.—The rights to surface support provided by ground water is inextricably related to the percolating water rights rules. Percolating waters are those which ooze, seep, filter, or percolate through the ground under the surface without a definite channel, or in a course that is uncertain or unknown and not discoverable from the surface without excavation. In the absence of competent evidence, all underground water is assumed to be percolating.

According to the English common law, percolating water is regarded as being as much a part of the freehold estate through which it courses or percolates as the clays, sands, gravel, soil, and rocks. In the absence of any malice or

any contractual or statutory restriction, the owner has the absolute right to intercept the water before it leaves his property and make whatever use of it he pleases, regardless of the effect that such use may have on an adjoining or lower proprietor through whose land the water infilitrates, percolates, or flows. An early California case adopted the common law English role which states that an individual may pump water from beneath his property without liability. The rule was articulated in *O'Leary Versus Herbert*. In this case, the defendents were mine operators who drove a tunnel in order to drain their mines. They encountered an unexpected flow of water in the core of the mountian and allowed the water to run off. As a result, the plaintiff's spring on the mountain ran dry and he sued for both an injunction and damages. In recognizing the common rule law, the California Supreme Court stated:

> The case on the facts is wanting in all of the elements upon which a recovery may be predicated. It falls fairly within the doctrine of *damnum absque injuria*.

Although *O'Leary Versus Herbert* implies that no cause of action will arise from doing a lawful act which harms another unless done with negligence or malice, the holding in the case was for the plaintiff because of the California rule governing the capture of percolating waters. This rule, first stated in the *Katz Versus Walkinshaw*, is that water rights are correlative so that each lawful user is entitled only to a reasonable share of the water common to all. The *O'Leary Versus Herbert* court believed that the use of water to drain a gold mine was not reasonable compared with the use by the spring owner for irrigation and domestic purposes. Thus, while the California Supreme Court endorsed the principle of *damnum absque injuria* it did not apply it to the use of ground water. *O'Leary Versus Herbert* left unclear whether the rule would be applied in cases of subsidence. The few early cases in other jurisdictions all endorsed the English common law rule.

Trend Is Toward Holding Ground-Water Pumpers Liable for Subsidence

The rule of law governing liability for subsidence caused by the removal of ground water is not settled in most jurisdictions although the trend appears to be toward greater liability. This change in the law is reflected by a reversal of the position of the American Law Institute in the restatement of Torts II compared with the restatement of Torts I.

The restatement of Torts I [Section 818 (1938)] stated the rule:

> to the extent that a person is not liable for withdrawing subterranean water from the land of another, he is not liable for subsidence of other's land which is caused by the withdrawal.

The position stated in the restatement of Torts II [Section 818 (1969)] is:

> One who is privileged to withdraw subterranean water, oil, minerals or other substances from under the land of another is not for that reason privileged to cause the subsidence of the other's land by such withdrawal.

United States Versus Anchor Oil.—*United States Versus Anchor Oil* was the largest damage suit in U.S. history for subsidence caused by pumping ground fluid. While the case never went to trial, it provided a great number of technical reports and legal briefs which shed light upon the problem of liability and damages for subsidence caused by the extraction of liquids. The case arose from damages to the Terminal Island Naval Base in California caused by subsidence which allegedly resulted from the pumping of oil from the Wilmington Oil Field.

Although substantial quantities of oil had been produced in the city of Long Beach for many years, it was not until 1938 that the Wilmington Oil Field was discovered beneath the tide lines and adjacent public and private upland in the harbor area of the city. This discovery led the City Board of Harbor Commissioners to enter into various agreements with oil companies whereby the latter acquired rights to take oil from the city property in the harbor district. Oil resources underlying privately owned land on the uplands were simultaneously developed by private interests.

In August 1958, the United States of America sued all the oil and gas producers in the field claiming that their operators had withdrawn underground support from its Naval Base on Terminal Island and other properties thereby causing subsidence which seriously damaged the government property. The goverment's complaint alleged production of oil began in 1932, that the subsidence of government property began in 1943 and continued at an increasing rate up to the time the damage action was filed.

By August 1957, an elliptically shaped bowl-like area of approx 41 sq km (16 miles) had subsided from 2 ft (0.6 m) on the periphery to a maximum of 24 ft (7 m) near the center. In addition, land surface was lowered to below the level of the adjacent San Pedro Bay and up to 9 ft (3 m) of lateral soil surface was displaced.

The plaintiff, the U.S. Government, argued that oil is a mineral like coal and thus the surface estate is entitled to subjacent support of the oil beneath the surface of the ground. The government also stated that oil was similar to asphalt which is also fluid mineral. It then argued that even if the percolating water rule rather than the mineral cases should govern the extraction of oil, that the majority of percolating water cases hold that liability attaches when subsidence is created by the withdrawal of ground water.

The government also rested its case on a nuisance argument. California Civil Code Section 3479 states, "Anything which is an obstruction to the free use of property so as to interfere with the comfortable enjoyment of life or property is a nuisance." However, the code also provides, "Nothing which is done or maintained on the express authority of a statute can be deemed a nuisance," and the pumping of oil in the Wilmington Oil Field was undertaken with express authorization by the state legislature. The government claimed while the oil pumping was authorized by statute, the subsidence was not.

Most of the defendents, which included the state of California, the city of Long Beach, and almost all of the major oil companies, admitted that the subsidence was caused by the withdrawal of oil. However, they disputed that they were liable for any damages caused by the subsidence. The defense was based upon the ingenious argument that the plaintiff claimed the right to have his oil laterally supported in place beneath his land by the pressure of defendant's oil, so that the plaintiffs' oil can support the surface.

The defendants claimed that the opposite is the rule of law. Federal rules of law give every landowner a right to take oil from his land, whether he drains his neighbor's land or vice versa. The same rules of law apply whether the oil is used for its mineral value or to support land.

The defendants claimed that one cannot apply the same rule of law to petroleum, gas, oil, and othe liquid minerals as the solid minerals because they migrate. Therefore, the plaintiff can claim no right to have the oil underneath its land stay in place.

The defendants further claimed that an individual does not really own the gas and oil beneath his land, but merely has a right to drill. They concluded that without malice or negligence, no liability for damages to an adjacent person's property can be assessed in the process of mining oil.

The defendants also asserted that cases which govern percolating water must also govern oil and that the Californian and American rule of law is that the liability for withdrawal for subjacent and lateral support is limited to harm done to the natural state of the land. No recovery may be had for structures when there is no negligence.

The case was settled out of court because the oil companies were enjoined from pumping oil from the Wilmington Field until the case was settled. Thus, they had a strong financial incentive to settle. Similarly, extensive litigation was of no advantage to the government who was willing to settle if the price was right.

The government was assured of no further subsidence by the passage of the Anti-Subsidence Act of 1958, which compelled oil producers in the Wilmington Field to unitize and undertake repressurization of the depleted reservoir. This legislation was enacted at the instigation of the city of Long Beach and upon expert advice that subsidence could be checked by injecting water to repressure the oil bearing sands. The statute clearly reflects a desire to retain the economic benefits of the Wilmington oil production, while relying on technology to prevent damage to private property rights. It provides a cause of action for property damage only where such damage results from a direct violation of an order or decision of the State Oil and Gas Supervisor. (The act shows the intent of the legislature to prevent further subsidence, but not to set liability.) Consequently, if there is an effective remedy for property damage caused by the Wilmington subsidence, it must be found in the common law principles applicable to lateral and subjacent support.

City of Los Angeles Versus Standard Oil of California.—Subsequent to the Anchor Oil case, and related to it, came *City of Los Angeles Versus Standard Oil Co. of California.* The city of Los Angeles had paid the entire bill for the damages caused by the failure of the Baldwin Hills Dam and sought contribution from Standard Oil Company.

The city of Los Angeles' claim was based on the following grounds. From 1924 until the time of the disaster, December 1963, the defendants extracted petroleum, water, and natural gas from the Inglewood Oil Field. In 1951, the plaintiff, the Department of Water and Power, built the Baldwin Hills Dam and Reservoir in the city of Los Angeles on a site adjacent to Inglewood Oil field. The drilling operations of the defendant, Standard Oil Company, caused the surface area of the oil field and surrounding property to subside. This subsidence caused the Baldwin Hills Dam to collapse, and the reservoir to

flood the surrounding area. The flood caused severe damage to the property of more than 3,700 property owners, killed at least five persons, and injured many others.

Since the litigation to determine the parties responsible for the losses would require an extended period of time and would frustrate prompt settlement of losses, the city paid a reasonable amount for the claims of those who were injured in the flood. The aggregate amount of the claims paid exceeded $12,000,000. The city claimed that it was not liable for the flood damage and the defendant oil companies were solely liable, but if the city was charged with any liability, it believed it should be held jointly liable for the flood damage with the defendant oil companies. The court agreed with the city and held that the defendant oil companies would have to contribute to the damage claims paid by the city.

Texas Cases.—Texas has suffered more damage from subsidence due to pumping ground water than any other state in the Union. Most of the subsidence has occurred in the Houston-Galvestone area. In Texas, the detrimental effect of subsidence has included: (1) Structural damages such as cracking buildings and disrupting pavements; (2) damages to well casings and buried utility lines; and (3) submergence of coastal lowlands. Texas has traditionally followed the English common law rule of liability. This rule was stated in *Houston and TCR Co. Versus East*, (1904) in regard to ground water, and in *Penny Versus Texas Gulf Sulphur Co.*, (1961) in regard to mining sulphur.

A landmark decision rendered by the Texas courts on the issue of subsidence caused by the removal of ground water occurred in *Coastal Industrial Water Authority Versus W. B. York*, (1976). Beginning in 1950, York's land (1.3 ha or 3.3 acres) began sinking into and was finally submerged into the Houston ship channel as a result of pumping water for commercial and industrial purposes.

The Water Authority intended to comdemn land on both sides of the ship channel to expand the port facility. The question raised in court was whether the Water Authority had to reimburse property owners on both sides of the ship channel for their land which was lost to the water due to subsidence. If the property owners did retain title to the submerged land, then they necessarily would be compensated by the Water Authority in the condemnation proceedings. The court held that the property owner did not lose title to the land due to the fact that it had become submerged from subsidence.

A recent case indicates that Texas may also be moving toward adopting the modern rule and holding ground-water pumpers liable for subsidence. *In Smith-Southwest Industries Versus Frenswood Developement Corp.*, (1978), a landowner brought a class action suit for other landowners in the area seeking recovery for subsidence of land caused by defendant's withdrawal of underground percolating water from wells located on defendant's land. The District Court granted summary judgment on the pleadings in favor of the defendants. The Appellate Court reversed and set a new precedent. The precedent was of short duration when the Texas Supreme Court reversed the Appellate Court and affirmed the judgment of the trial court. Both the majority and dissenting opinions of the Supreme Court, however, hold some promise for future consideration of negligence in the pumping of ground water. The plaintiffs, who owned land along the west bank of the Galveston Bay, including some shoreline, contended that the massive pumpage of subsurface water to sell to industrial customers

proximately caused the sinking and loss of elevation above mean sea level of their property. Flooding resulted from the subsidence and caused damage and destruction to the land and improvements.

These wells were drilled from 1964 through 1971, even though previous engineering reports to defendants showed the pumping would result in land subsidence in the area. One defense raised by the defendants was the contention subsidence was a pre-existing problem in the area and that other owners of other wells in the area contributed to the subsidence. Consequently, defendant filed action against 32 companies and municipalities alleging they contributed to the problem.

The Appellate Court stated that Texas follows the English common law and there is unlimited right to pull water from underneath one's land and there is no liability for damage to another's land. The court, however, noted that the principle of the use of the land to pump water is in direct conflict with another rule; namely, that a property owner must use his property so as not to injure that of another and that land ownership itself creates a duty to use reasonable care in the exercise of property rights incident to such ownership.

The Appellate Court did not reverse the English rule but held that there was responsibility for subsidence due to pumping on the theories of negligence and nuisance.

The Texas Supreme Court, in reversing the Appellate Court and affirming the decision of the Trial Court, stated plaintiffs were in effect contending that the "reasonable use" doctrine should apply to ground water the same as it does to other real property. The court further noted that in *East Versus Houston & T. C. Ry. Co.* (1903), it had chosen against "reasonable use" rule and for "absolute ownership" rule. It further noted only waste and malice were possible limitations to the rule.

The Court based its opinion on the restatement of Torts, Section 818 (1939) as representing the subject as pronounced in common law jurisdiction; and the adoption of the common law of England by Texas in 1840. It further denied that mineral extraction law should be applied to the case.

The decision is tempered by a statement that the court conceived its duty to apply property law as it existed at the time of the actions complained in the suit even though it disagreed with certain aspects of the existing rule. In commenting on future wells and subsidence, the court noted the legislature has entered the field of regulation of ground water withdrawals and subsidence. It further stated the

> Legislature has assumed it proper role, because our courts are not equipped to regulate ground water use and subsidence on a suit-by-suit basis. . . . Our consideration of this case convinces us that there is no valid reason to continue this special immunity insofar as it relates to future subsidence proximately caused by negligence in the manner which wells are drilled or produced in the future.

The court did indicate that as to future subsidence caused by wells drilled or produced in the future, it would recognize and apply the law of negligence along with willful waste and malicious injury as limitation on the present rule applicable to subsidence resulting from pumping. It is not clear if the court

would delay in accepting subsequent cases thereby giving legislatively created agencies a chance to solve the problem.

In 1975, the Texas Legislature created the Harris-Galveston Coastal Subsidence District with power to provide for spacing of water wells and regulate production of water wells. In the same act, the legislature confirmed private ownership of underground water subject only to the regulations of the district.

The Minority Opinion in this case would affirm the judgment of the Appellate Court. It bases its opinion on the landowner's right to subjacent support for their land. The theory put forth is that pumping of ground water reduced pore pressure, thereby causing collapse of supporting clay layers and resulting in subsidence, a result which was both predictable and known by defendants. The Minority Opinion upholds the theory of negligence as actionable and disagrees with the Majority Opinion on future cause noting pumpers will probably feel benignly toward those who must suffer the loss of their lands. They conclude by noting it is unfair to recognize the plaintiffs possess an action but then denying them the remedy.

Two recent Wisconsin cases find liability for subsidence damage caused by pumping of underground water. *State Versus Michaels Pipeline Construction Co.*, (1974), and *State Versus Deetz*, (1974).

Current Problems of Subsidence and Solution of Ground-Water Basin Management

Fixing liability on the ground-water pumper for subsidence of the land is an important step in discouraging negligent use of ground-water resources. However, the placement of liability on the ground-water pumper has little deterrent effect when all of the overlying users are pumpers, and in common are responsible for the subsidence. Thus, subsidence and depletion of the ground water, is subject to what has been called "the tragedy of the commons." This occurs when overlying owners calculate that the lowered water table from a steady-state condition will be less of a disadvantage to them than the added benefit of developing more water.

Status of Subsidence Problems.—Subsidence in the Santa Clara Valley of California was halted by importation through the State Water Project of additional water in the early 1960s. Increasing water demands during the 1960s and 1970s are being offset by additional imported water, which, along with conservation and reuse, should satisfy demands until the year 2000.

Overdraft of the San Joaquin Valley ground water and accompanying subsidence are continuing and were intensified during the recent drought. The large size of the area affected, and its possible effects on stream location, may make this situation more critical in the future.

Ground Water Management Programs.—A recent report by the Governor's Commission to Review California's Water Rights Law suggests an innovative approach to the management of ground-water resources. Currently there are no comprehensive ground-water management programs at the state level. However, there are several local ground-water management programs.

The Orange County Water District has been a leader in ground-water management. It has a wide variety of powers and has exercised them in an integrated

approach to management of the basin. The district has levied a pump tax which applies to all ground-water extraction, and does not provide advantages to early appropriations or overlying uses. While the tax applies to all users, it is not final and may be increased or decreased in order to affect the relative amounts of ground water and surface water that are used.

The district is also involved in the importation of water to replenish ground-water supplies. This water is spread in areas selected because they allow the water to percolate into the ground-water basin, or is provided as surface water in particular areas to allow the ground-water level to recover. In addition to its other functions, the district requires pumpers to file periodic water production statements with it.

In contrast to the Orange County system, the San Gabriel Valley manages the ground-water basin by a combination of an adjudication and a watermaster. The San Gabriel watermaster is composed of nine members appointed by the court pursuant to an agreement among the ground-water users in the adjudicated area. The watermaster is a policy maker with broad powers. Not only can the watermaster levy a charge on pumping in excess of a pumper's adjudicated share of the basin's yield, but it can conduct a ground-water replenishment program, and control storage of water in the basin.

The adjudication approach includes a judgment which allocates an amount of water to parties which they believe to be fair and reasonable. It is the responsibility of the watermaster to manage the available water and to ensure each party gets an appropriate share.

In view of the success of local ground-water management programs, the Governor's Commission to Review California Water Rights Law developed model legislation based upon local control of ground-water basins. The findings of the proposed legislation state: Ground-water resources have not generally been protected and managed to the same extent as surface resources, even though ground-water resources supply nearly half the water applied to the state in normal years, are an emergency supply in drought years, and are extremely important for water storage, treatment, and distribution. Ground-water resources management can be most effectively administered by local agencies within a framework of statewide policy. The state must be prepared to exercise its full power and jurisdiction to protect the primary interest of the people of the state in the protection, management, and reasonable beneficial use of ground-water resources if local entities do not adequately manage those resources.

Under the proposed legislation, local entities include: a city, county, public utility, water company, water district, a water replenishment district, a joint powers authority, such as a regional district, or a watermaster. The State Department of Water Resources is to cooperate with local agencies in gathering data about overdraft and subsidence problems. After the problems have been identified, the State Water Resources Control Board selects, with cooperation of local government, an appropriate local agency to manage the ground water basin. If no appropriate agency is forthcoming the board has power to form a ground-water managment district.

Subsequent to selection of a ground-water management authority, the authority must adopt a ground-water management program for the area and transmit it to the Water Resources Control Board for comment. The ground-water management program must consider the following factors:

1. The hydrological and geological characteristics of the ground-water management area.
2. Present ground-water and surface water management programs and the status of any adjudications or contractual arrangements affecting water supply and delivery.
3. The present and probable future availability of supplemental water supplies.
4. Present and probable future reasonable and beneficial uses of water.
5. The ground-water conditions that could reasonably be achieved through alternative management program, including mitigation measures which could be taken to minimize any significant adverse environmental impact.
6. The economic consequences of alternative management programs, including the effect of limitations on extraction.
7. The probable future condition of ground-water resources with no additional ground-water management and the economic consequences of no additional management.

The proposed legislation provides the ground-water management and the economic consequences of no additional management.

The proposed legislation provides the ground-water management authority with broad powers to implement the ground-water management program. Without being limited to the following acts, a ground water management authority may:

1. Provide for the conjunctive use of ground-water and surface water resources with the ground-water management area.
2. Store water in and recapture water from surface reservoirs or ground-water basins within or outside of the ground-water management area.
3. Regulate the storage of water and the use of ground-water basin storage space in ground-water basins within the ground-water management area.
4. Acquire water and water rights within or outside of the ground-water management area.
5. Purchase and import water into the ground-water management area.
6. Conserve and reclaim water within or outside of the ground-water management area and require conservation practices and measures within the area.
7. Buy and sell water and water rights at such rates as shall be determined by the board of directors.
8. Exchange water and water rights.
9. Export water and control the export of water from the ground-water management area.
10. Limit extraction to respond to conditions of long-term overdraft, subsidence, water quality and other significant environmental degradation, well interference, or the threat of any of the foregoing.

The California Commission on Water Rights should be commended for its bold program to solve the state's problem of ground-water overdraft and subsidence. Unfortunately, the proposed legislation has not moved through the state legislature. This is because of the intense opposition of farming interest throughout the state. Farmers view the legislation as creating more bureaucracy, restricting their actions with more regulation, and ultimately costing them more for water. While the proposed legislation will have economic impact on the

farm community, the cost of postponing action on protecting the state's ground-water basins and preventing further subsidence will eventually be far greater. In areas when the salt deposits build up in soil and where an inland salt lake may be created by subsidence, the damage could be irreversible.

Summary

Subsidence of land due to overdraft of ground water is a relatively recent phenomenon. The early English and American courts first held that the ground-water extractor was not responsible for the subsidence of his neighbor's land. In recent years, the courts have changed their policy and now tend to hold the pumper of the ground water liable. While in some cases it has been held that landownership creates a duty to use reasonable care in the exercise of property rights, other cases cite theories of negligence and nuisance as the basis for fixing liability for subsidence damage. Some cases have restricted damage claims to the value of land in its natural state while others have included loss of improvements.

The responsibility for control of land subsidence due to pumping may eventually rest with water agencies. This will occur as the demand for water exceeds supply and management of ground-water basins comes under increasing government oversight and control.

Opinions expressed herein are not necessarily those of the writers' agency.

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ECONOMIC CONSEQUENCES OF LAND SURFACE SUBSIDENCE[a]

By Lloyd C. Fowler,[1] F. ASCE

The Santa Clara Valley in Santa Clara County, Calif., is located at the south end of San Francisco Bay. The primary source of water for this area has been the underlying ground-water basin. It was developed as a major source of water in the late 1800s and indications of overdraft were obvious in the early 1900s. At that time artesian wells were beginning to stop flowing and ground-water levels had fallen lower in many parts of the basin. The insidious resultant effect of overdraft of this ground-water basin was not noted until 1932 when releveling of a benchmark set in 1912 indicated a loss in elevation of 1.2 m. The valley floor was subsiding.

In 1934 a valleywide network of benchmarks was established. The elevation of these benchmarks has been resurveyed a dozen times between 1934 and 1967 to determine the amounts of land surface subsidence. During this 33-yr period, land surface subsidence has ranged from 0.3 m–1.2 m under southern San Francisco Bay to 2.4 m in central San Jose (Fig. 1). These surveys show the area affected by land surface subsidence to be about 63,000 ha.

Studies by the U.S. Geological Survey have shown that the subsidence of the land surface is caused by the overdraft and the resulting decline of the artesian pressure head in the ground-water basin (Fig. 2). This decline in subsurface hydraulic pressure allows an increase in the effective overburden load on the deeper materials comprising the valley-fill alluvium. These sediments compact under the increasing load and the land surface sinks. Most of the compaction occurs in the fine-grained silt-clay beds which are the most compressible. Once the water is squeezed out of these fine-grained beds, and the materials compacted, it is not possible to reexpand them to recover the lost elevations.

The land surface subsidence is a gradual lowering of a large area in such a manner that it is not apparent to the general observer except near the bay. There, the ocean provides a ready reference point. Land that was once above the high tides is now below the high tide and even below mean sea level.

[a]Presented at the October 22–26, 1979, ASCE Annual Convention and Exposition, held at Atlanta, Ga.

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0151/$01.00.

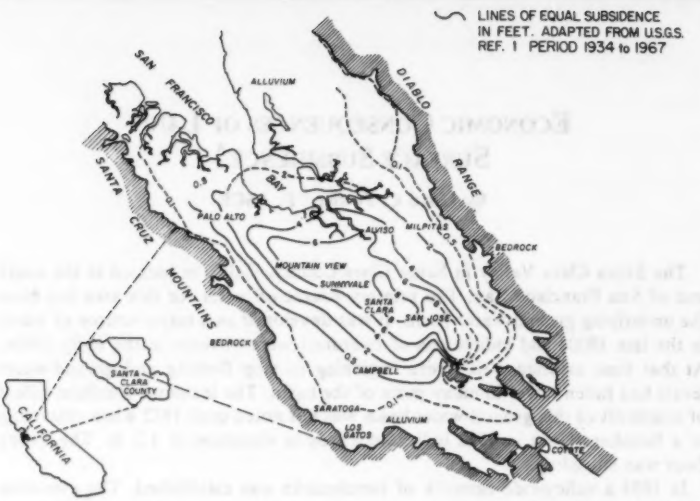

FIG. 1.—Subsidence Contours in North Santa Clara Valley

At present, about 5,000 ha of northern Santa Clara Valley are subject to salt-water flooding from the high tides in San Francisco Bay. Once a person standing on the land near the bay could look down upon the waters. Now, a person standing on the same spot looks up at levees keeping the salt water off the land (Fig. 3).

Effects of Land Surface Subsidence

As previously mentioned, land surface subsidence in Santa Clara Valley affects a very large area, and, therefore, the changes in slopes of the land surface are very gradual. They are not noticeable except with precise surveys. There are no differential disruptions which may have caused cracking of the ground surface. Nevertheless, the changes in grades, while gentle, have had adverse effects on sewerage, drainage, and flood control systems.

The compaction of subsurface materials that results in land surface subsidence occurs primarily in the finer grained silt-clay formations. The rearrangement of the fine-grained material reduces the pore space and drives out water previously stored in the spaces. It is estimated that during the 33-yr period from 1934 to 1967 the volume of water driven out of these pore spaces, as represented by the change in elevation of the land surface, was about 617,000,000 m^3. This additional water was squeezed into the aquifer systems and extracted through wells for use on the overlying lands. Land surface subsidence does not appear to have affected either the yield or the storage capacity of the coarse-grained aquifer zones.

There have been no known structural damages to buildings or other surface

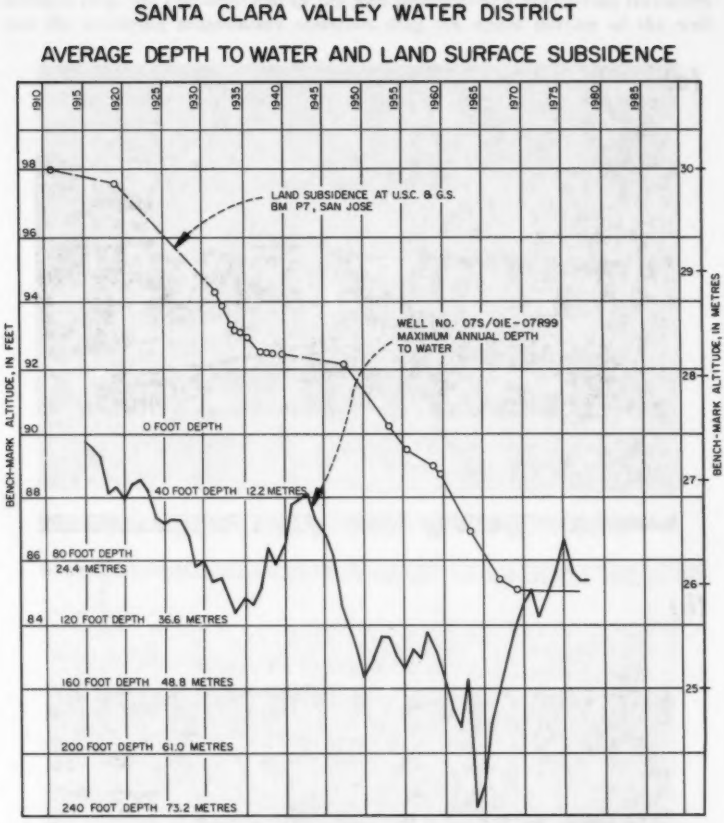

FIG. 2.—Average Depth to Water and Land Surface Subsidence

facilities as the result of land surface subsidence. The change in slope of the land surface has been so gradual that there is no apparent tilting affect on even the taller buildings. Buildings and other surface features are, however, subject to more potential flooding as a result of the lowered elevations. This flooding is both from freshwater streams and from high tides in San Francisco Bay. Another adverse affect of land surface subsidence on structures is the loss of view at the bay (Fig. 3). This loss of aesthetics, while its value cannot be measured, is considered by many to be a major adverse affect of land surface subsidence in Santa Clara Valley.

Most large, producing water wells extend to depths of over 125 m. The zone of most active compression appears to be about 60 m–100 m below land surface. As a result of the compression in these zones, water well casings buckle or

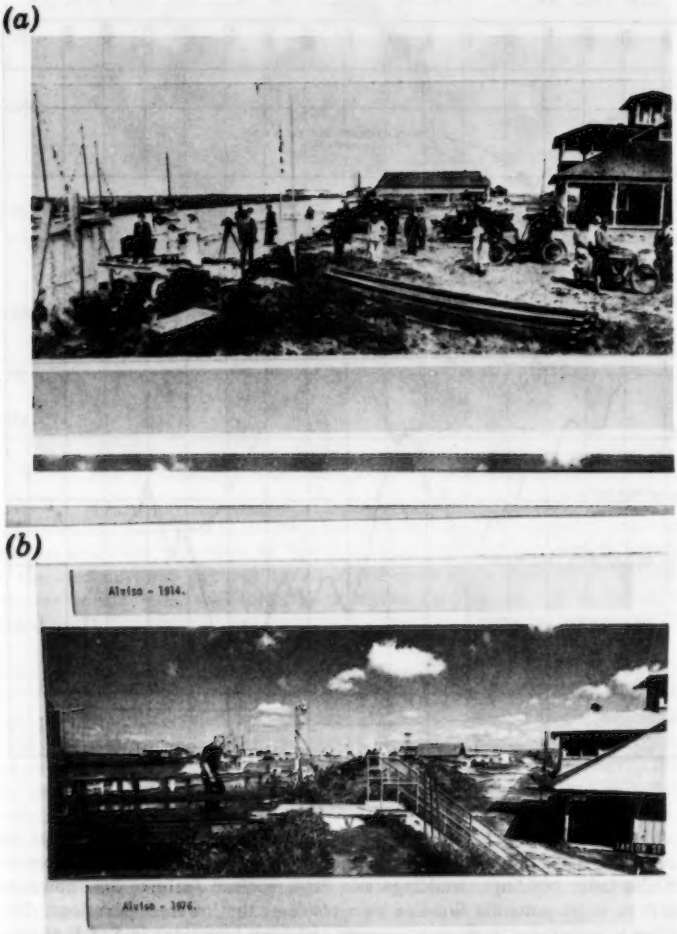

FIG. 3.—South Bay Yacht Club (Building at Right) in Alviso: (*a*) In 1914; and (*b*) In 1976

collapse (Fig. 4). The deep end of the well casing is in a nonmoving formation and the overlying sedimentary materials drag the upper portion of the well

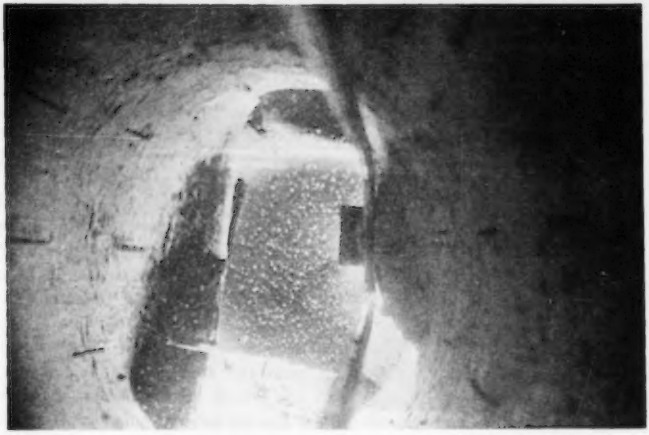

FIG. 4.—Water Well Casing Buckled by Land Surface Subsidence

FIG. 5.—Railroad Flooded by High Tides in San Francisco Bay as Result of Land Surface Subsidence

casing down as the land surface subsides. This compressional force on the casing is, in many instances, enough to cause the casing to collapse or to buckle to such an extent that the well fails. The buckled or collapsed casings must

FIG. 6.—Highway Bridges Across Calabazas Creek; Eastbound Lanes in Rear Raised to Clear Flood and Tide Water Levels in Channel, Westbound Lanes in Front Subject to Flooding as Result of Land Surface Subsidence

FIG. 7.—Storm Drainage Pumping Station at Guadalupe River; This Land Formally Drained by Gravity Now Must Be Drained by Pumping as Result of Land Surface Subsidence

be repaired or a new well drilled at considerable expense. There have been instances where the skin friction of the overlying sediments on the well casing is inadequate to compress the casing and instead the well merely protrudes out of the ground.

There are numerous examples of changes in grades of sanitary sewers caused by land surface subsidence that has reduced their flow carrying capacity. Again, the changes in grades are gradual and do not adversely affect the structural integrity of the sewer. The loss of flow capacity is made up by either enlarging the sewer, putting an additional parallel unit into service, or installing a pumping station. This has been necessary only near the bay where grades are very flat.

Roads and railroads near the bay have been subject to additional flooding by both freshwater and salt water as a result of land surface subsidence. The flooding causes disruption of traffic and rapid deterioration of subgrades (Fig. 5). Where these transportation facilities cross stream channels entering the bay, their grades have to be raised so that they can clear the raised levees and the higher water surface profiles in the channels (Fig. 6). Because of the flatter hydraulic slopes, flood control channels are enlarged to maintain the same carrying capacity, and this requires longer bridges for the transportation facility.

Land surface subsidence has resulted in placing large areas of the valley below high tide and even below sea level near the bay. In order to prevent this area from being flooded by salt water, the bay has to be ringed by levees. Not only does the area have to be protected against salt-water flooding, but the area must also be protected against freshwater flooding from inland upstream watersheds. As the land surface subsides, the water elevation in the flood control channels leading to the bay becomes higher in relation to the land. These channels must be leveed and existing levees raised in order to contain the flood flows from the upstream areas. The higher water surface elevation in the flood control channels requires that all local storm drainage in the areas near the bay be pumped into the outfall channels (Fig. 7). The construction and the operation and maintenance of these storm drainage pumping stations are an added burden on the adjacent areas.

Costs of Land Surface Subsidence

The costs resulting from the loss of the aesthetic views of the bay brought about by land surface subsidence are unknown, but in the minds of many it is one of the larger adverse affects imposed upon the shoreline communities.

Records from municipal water utilities, private water companies, and water well drillers show that during the 5-yr period, 1960–1965, over 250 wells were damaged by land surface subsidence. Since these records represent less than one quarter of all the wells in the subsiding area, the number of damaged wells must be at least 1,000. The cost of repair exclusive of those that had to be replaced ranged from $500 to $5,000 and averaged about $2,000. This represents a direct cost resulting from land surface subsidence of $2 million during the 5-yr period, exclusive of the cost of any replacement wells. This also does not consider the costs of repair of wells damaged by land surface subsidence in the years prior to the 5-yr survey period.

Sanitary sewerage systems adversely affected by land surface subsidence have been enlarged or paralleled or have had pumping stations installed. All the

effluent from the sewage treatment plants in northern Santa Clara Valley must be pumped up into the Bay as the result of land surface subsidence. The costs of modifications on the sewerage systems have exceeded $1,000,000. This is exclusive of the operation and maintenance costs of the required pumping facilities.

The raising of grades for transportation facilities and the enlargement or replacement of bridges as the result of land surface subsidence have totaled over $2,800,000. This is exclusive of higher maintenance costs of damaged roadbeds as the result of flooding.

The existing bayfront levees protecting the lands from saltwater flooding are considered inadequate. A 1973 study of the salt-water flooding problem estimated that the most economical alternative solution was to construct a new $58,000,000 levee system. The raising of channel levees to meet the higher water levels in relation to the lowering land surface cost over $10,000,000 by 1979 with

TABLE 1.—Costs of Land Surface Subsidence Santa Clara Valley

Item (1)	Cost, in dollars (year) (2)	1979 costs, in dollars (3)
Water well repair	2,000,000 (1975)	7,600,000
Sanitary sewers	1,000,000 (1970)	2,400,000
Transportation-bridges	2,800,000 (1975)	4,100,000
Bay levees	58,000,000 (1973)	103,000,000
Channel levees	10,000,000 (1979)	10,000,000
Drainage pumps	2,700,000 (1975)	4,000,000
Total		131,100,000

more work yet to be accomplished. The pumping stations constructed by local jurisdictions to service local storm drainage have cost about $2,700,000. Some areas still need to be serviced by storm drainage pumping systems. This cost has not been estimated.

These direct costs attributable to land surface subsidence total more than $130,000,000 when brought to present 1979 values (See Table 1). They do not include extended operation and maintenance costs of the pumping facilities. That annual cost may exceed the total capital cost outlay over the next few decades.

Conclusion

Water imported through the South Bay Aqueduct of the State Water Project became available to northern Santa Clara Valley in 1965. The advent of this additional water eliminated the ground-water overdraft, and the replacement

of the vacated storage space underground began. The storage of additional water underground raised the ground-water levels and increased the artesian pressures to the point where land surface subsidence ceased for all practical purposes in 1969. Since that date, no significant amount of land surface subsidence has been recorded. The annual costs to repair damages resulting from land surface subsidence have been reduced. Considerable funds are still being spent to repair the damages resulting from the periods of most active land surface subsidence, but as long as the ground-water levels can be maintained at or above those of 1969, increases in damages and resultant repair costs will be eliminated. Therefore, it is essential that imported water supplies continue to be available in northern Santa Clara Valley in order that the ground-water levels will not drop below the critical elevations of 1969 and the water needs of the area can continue to be met.

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SUBSIDENCE IN HARRIS AND GALVESTON COUNTIES, TEXAS[a]

By Ronald J. Neighbors[1]

INTRODUCTION

Land surface subsidence has been described as a result of compaction of fine-grained material in the subsurface. Compaction is caused by loading due to pressure declines associated with the removal of subsurface fluids, principally, water, oil, and gas. (3).

Land surface subsidence has become critical to parts of the Houston-Galveston Region of Texas. Some low-lying areas along Galveston Bay are subject to inundation by normal tides, and an even larger part of the region may be subject to catastrophic flooding by hurricane tides.

House Bill 552, Chapter 284, Acts of the 64th Texas Legislature, 1975, created the Harris-Galveston Coastal Subsidence District, a two-county regulatory agency

> . . . to provide for the regulation of the withdrawal of groundwater within the boundaries of the district for the purpose of ending subsidence which contributes to or precipitates flooding, inundation, or overflow of any area within the district, including without limitation rising waters resulting from storms or hurricanes.

DEVELOPMENT OF GROUND WATER IN HOUSTON-GALVESTON REGION

In order to understand the subsidence problem in the Houston-Galveston area, it is helpful to describe the historical development of ground water. Pumpage has been inventoried for many years by the U.S. Geological Survey by areas which are defined by the predominant type of water use. Fig. 1 shows the principal areas of ground-water withdrawal. Fig. 2 shows average rates of pumpage for the past 5 yr in principal areas of ground-water withdrawal in the region.

[a]Presented at the October 22–26, 1979, ASCE Annual Convention and Exposition, held at Atlanta, Ga.

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1981. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0161/$01.00.

Houston Area.—In 1887, when the city of Houston purchased a private water supply company, the demand for water was 1 mgd–2 mgd (0.04 m^3/s–0.09 m^3/s). The demand grew steadily and, at the end of 1972, the Houston Water Department used 164 mgd (7.16 m^3/s) of ground water and about 58 mgd (2.5 m^3/s) of treated surface water. Prior to the time Houston began using surface water in 1954, it was the largest city in the nation utilizing only ground water for public supply.

In 1973, the Houston Water Department increased the use of surface water to 63 mgd (2.7 m^3/s) and decreased the use of ground water to 156 mgd (6.83 m^3/s). By 1978, the city had increased the use of surface water to 112 mgd

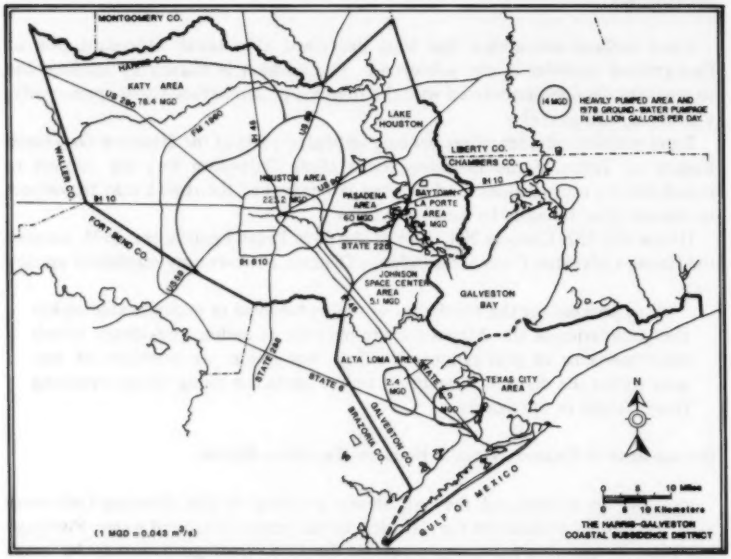

FIG. 1.—Major Areas of Ground-Water Withdrawals and Average Rates of Pumping in 1978

(4.85 m^3/s), but due to the overall growth in population and water demand, increased its use of ground water to 196 mgd (8.49 m^3/s). During 1978, an additional 32 mgd (1.4 m^3/s) of treated surface water from the East Treatment Plant was sold to municipal and industrial customers.

Pasadena Area.—Pumping of ground water for industrial use in the Pasadena area began near the end of World War I and grew steadily until 1936, when annual pumpage was about 15 mgd (0.65 m^3/s). The amount of surface water used was less than 20 mgd (0.87 m^3/s) when Lake Houston was completed in 1954. In 1953, 87 mgd (3.8 m^3/s) of ground water was used in the area. In 1972, 120 mgd (5.26 m^3/s) of ground water and 82 mgd (3.6 m^3/s) of surface

water was used. In 1972, about 104 mgd (4.50 m^3/s) of ground water was used for industrial use. By 1975, increased efforts to conserve industrial water and reduce wastewater discharges had decreased the use of ground water by industries to 94 mgd (4.1 m^3/s). In 1976, industries began to use surface water from the Trinity River and Lake Livingston, imported through the Coastal Industrial Water Authority delivery system. Total ground-water use in the Pasadena area was reduced to 60 mgd (2.6 m^3/s) during 1978, and industrial ground-water use was reduced to 43 mgd (1.9 m^3/s).

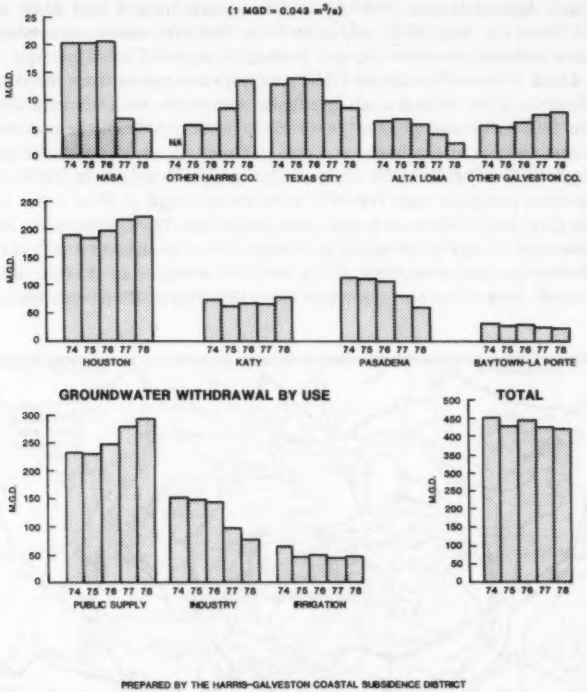

FIG. 2.—Ground-Water Withdrawal by Area

Katy Area.—All water used in the Katy area is ground water and, until recent urbanization impacted the area, more than 80% of the water pumped was for rice irrigation. Groundwater pumping increased from about 1 mgd (0.04 m^3/s) in 1893 to about 16 mgd (0.70 m^3/s) in 1946, and then rapidly increased to about 160 mgd (7.01 m^3/s) in 1954. In 1978, Katy Area (Harris County only) pumpage was 78 mgd (3.4 m^3/s), with 47 mgd (2.1 m^3/s) for rice irrigation. Between 1974 and 1978, ground-water use for public supply increased from about 5 mgd (0.2 m^3/s) to over 22 mgd (0.95 m^3/s).

Baytown-La Porte Area.—Ground-water pumping from large capacity industrial wells in the Baytown-La Porte area began about 1918. Pumping rates increased from about 5 mgd (0.2 m^3/s) in 1919 to about 32 mgd (1.4 m^3/s) in 1972. In 1972, 24 mgd (1.1 m^3/s) of ground water pumped was for industrial purposes. In 1978, industrial pumpage was reduced to 10 mgd (0.43 m^3/s) due to increased surface water use. Surface water conversions planned for 1979–1980 should reduce withdrawals in the area by about 8 mgd (0.4 m^3/s).

NASA Area.—With the development of the Lyndon B. Johnson Space Center and the Bayport Industrial Area, groundwater use in the National Aeronautics and Space Administration (NASA) area increased from 1 mgd (0.04 m^3/s) in 1960 to about 21 mgd (0.91 m^3/s) in 1976. Surface water conversions since 1976 have reduced ground-water use to about 5 mgd (0.2 m^3/s).

Alta Loma Area.—The city of Galveston began pumping from the Alta Loma Well Field in 1894. Withdrawals gradually increased from about 2 mgd (0.09 m^3/s) in 1896 to 13 mgd (0.57 m^3/s) in 1972. In August 1973, the use of surface water from Lake Houston began at a rate of 6 mgd (0.3 m^3/s) and was increased to 8 mgd (0.4 m^3/s) in 1976 and to 10 mgd (0.43 m^3/s) in 1977. Average ground-water pumpage rates for 1978 were about 2 mgd (0.09 m^3/s).

Texas City Area.—Ground-water pumping in the Texas City area increased from less than 2 mgd (0.09 m^3/s) in 1930 to about 30 mgd (1.3 m^3/s) in 1940. Withdrawals decreased slightly at the end of World War II, then decreased rapidly after 1948 when surface water from the Brazos River was brought into

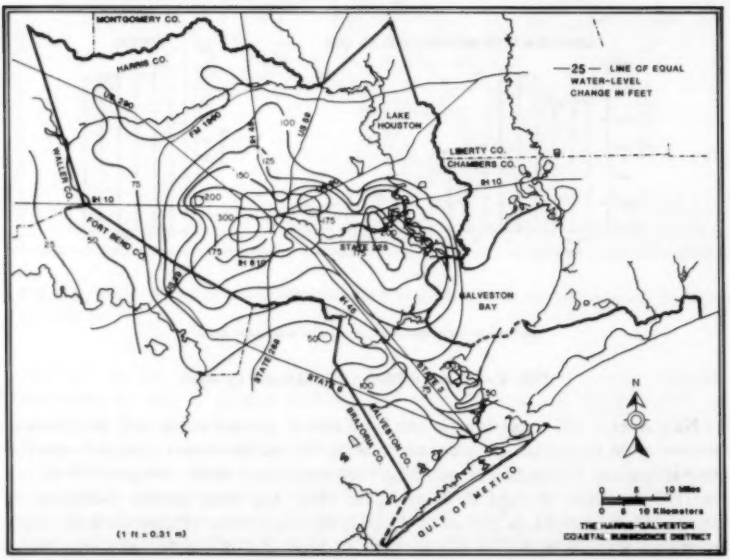

FIG. 3.—Approximate Water-Level Changes in Chicot Aquifer, 1943–1973

the area. Ground-water withdrawals gradually increased to 14 mgd (0.61 m^3/s) in 1972. Surface water conversions by two major industries reduced withdrawals to 9 mgd (0.4 m^3/s) in 1978. Municipal conversions to surface water should reduce pumping by an additional 5 mgd (0.2 m^3/s) by 1980.

Water Level Declines

As a result of large amounts of water having been pumped from the ground, the pressure in the artesian aquifers has declined. This decline in pressure, reflected by lower water levels in wells, is the principal cause of regional land-surface subsidence. Fig. 3 shows the decline in water levels for 1943–1973 in wells tapping the Chicot Aquifer, and Fig. 4 shows the declines in water levels for the same period in wells tapping the Evangeline Aquifer. In the Pasadena and Baytown-La Porte areas where ground water withdrawals are heavily concentrated, the decline of water levels in wells completed in the Chicot Aquifer was about 200 ft (61.0 m) during 1943–1973. Water levels and wells completed in the Evangeline Aquifer declined as much as 325 ft (99.1 m) between 1943 and 1973 (3).

Recent reductions in ground-water pumpage have caused water level rises in both the Chicot and Evangeline Aquifers. Measurements, in wells completed in the Chicot Aquifer between 1977 and 1979, show rises of up to 100 ft (30.5 m) throughout Galveston County and southeast Harris County. Water level

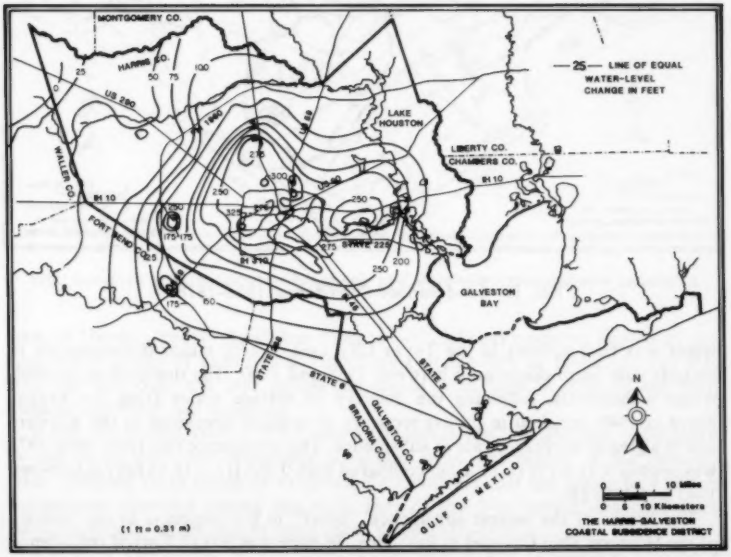

FIG. 4.—Approximate Water-Level Changes in Evangeline Aquifer, 1943–1973

declines in excess of 10 ft (3.05 m) in the Chicot Aquifer were observed in southwest Harris County. The same pattern of water level changes were found in the Evangeline Aquifer from 1977 to 1979, with rises of up to 100 ft (30.5 m) in the Pasadena-Deer Park area and declines in excess of 50 ft (15.2 m) in the southwest part of Harris County (4).

Compaction and Land Surface Subsidence

Fig. 5 shows the total amount of subsidence in the Houston-Galveston Region for 1906–1978. Land-surface subsidence resulting from the pumping of ground

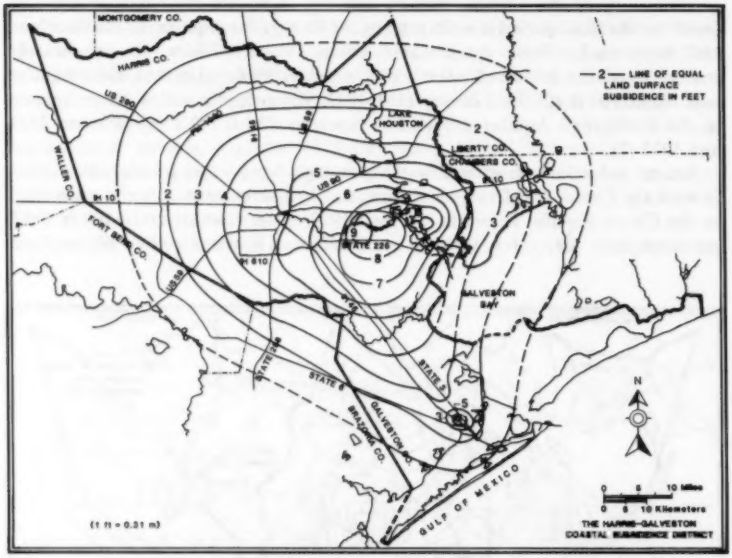

FIG. 5.—Land-Surface Subsidence, 1906–1978

water was first noticed in the Texas City area, where minor discrepancies in altitude data were discovered between 1938 and 1940. The decrease in groundwater withdrawals, following the delivery of surface water from the Brazos River in 1948, resulted in partial recovery of artesian pressures in the aquifers and in a greatly decreased rate of subsidence. The subsidence rate from 1964–1973 was approx 0.11 ft/yr (1.1 nm/s) compared with 0.366 ft/yr (3.54 nm/s) between 1940 and 1952 (3).

The center of the largest subsidence "bowl" in the region is in the vicinity of the Houston Ship Channel at Pasadena. In excess of 9 ft (2.7 m) of subsidence occurred between 1906 and 1978 or an average rate of 0.125 ft/yr (1.21 nm/s). Subsidence during some years, however, approached 0.4 ft/yr (3.8 nm/s).

The area of active subsidence is expanding. Between 1943 and 1954, about 350 sq miles (907 km^2) had subsided 1 ft (0.3 m) or more. By 1973, 2,500 sq miles (6,475 km^2) had subsided 1 ft (0.31 m) or more. By 1978, more than 4,500 sq miles (11,655 km^2) had subsided 1 ft (0.31 m) or more.

A $750,000 releveling survey was begun in the Spring of 1978 by the National Geodetic Survey with the Subsidence District serving as local sponsor. The 1973–1978 subsidence pattern is similar in many respects to subsidence prior to 1973 in that the area of greatest subsidence was 1.2 ft (0.37 m) and located in the Pasadena-Deer Park area on the south side of the Houston Ship Channel. Contrasted to prior history, however, is the increase in subsidence in the western

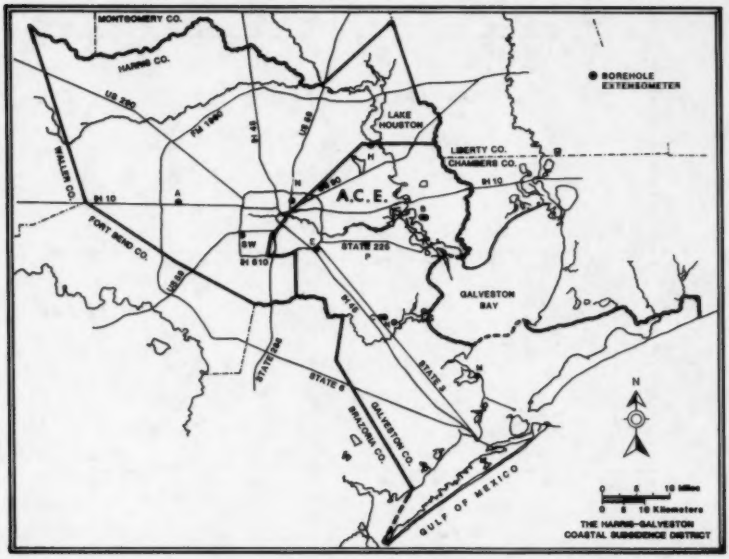

FIG. 6.—Area of Concentrated Emphasis and Borehole Extensometer Locations

part of Harris County. Small but definite 1-ft (0.31-m) contours appear at the southwest corner of loop 610 and northwest of the city of Houston. The 1-ft (0.31-m) loss over the 5-yr period represents from 25% to 35% of the total subsidence which has occurred.

While the comparison of 1973 and 1978 elevations indicate that subsidence is continuing and even increasing in certain areas, the results of the monthly data collected from borehole extensometers indicate that the major part of the subsidence occurred during the early part of the 5-yr period and that rates have drastically decreased in most areas since 1975–1976.

The U.S. Geological Survey collects compaction and subsidence data on a monthly basis from ten borehole extensometers (compaction monitors) located

at eight locations in the two-county area. The location of each of these sites is shown on Fig. 6. The monitors at Clear Lake City, Pasadena, and Addicks are of sufficient depth to record total subsidence. Also shown in Fig. 6, are

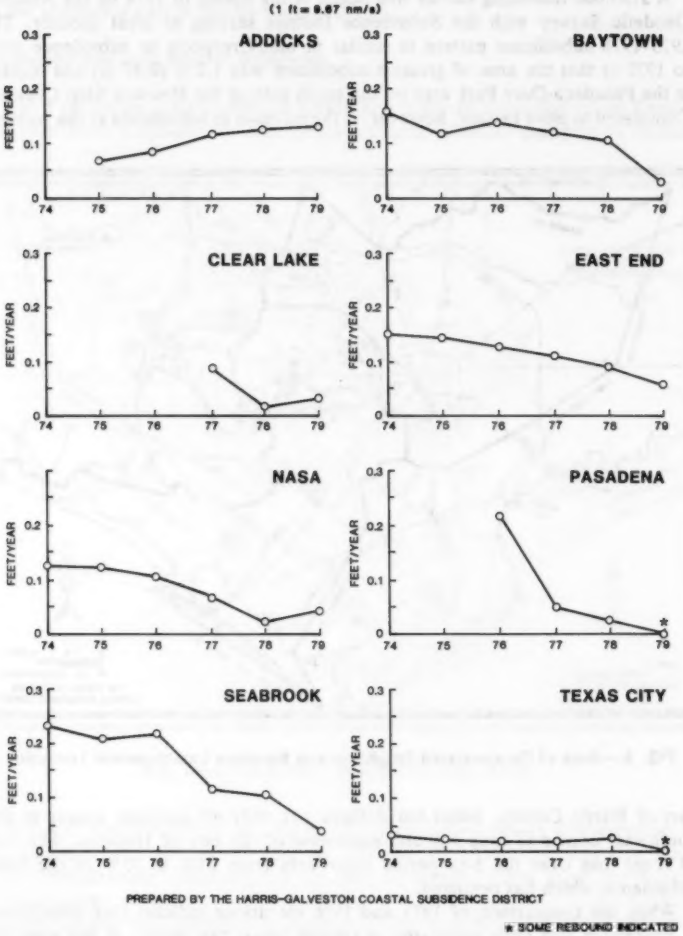

FIG. 7.—Borehole Extensometers

the locations of three additional sites where monitoring facilities are currently under construction at a total cost in excess of $880,000.00. Each site was selected

because of changes in pumping patterns and will contain one subsidence monitor and four piezometers.

Fig. 7 is a series of graphs depicting the annual rate of subsidence at each of the eight compaction monitor sites. The rate of compaction is decreasing at all sites, except Texas City where it remains relatively constant and at Addicks where it is increasing. The declines in compaction rates are in the areas in which ground-water withdrawals have been curtailed. The increase in the compaction rate at Addicks reflects recorded increases in withdrawal in the Houston and Katy areas.

Economics of Subsidence

Subsidence in the region has resulted in permanent inundation or increased exposure to flooding of land, caused well casings to collapse, lessened the efficiency of storm drainage facilities and, in the opinion of many, contributed to the activation of geologic faults. Since 1943, several thousand acres of bayfront property have been submerged into the tidal reaches of Galveston Bay. Uncontrolled subsidence would increase the amount of land submerged to a total of 20,000 acres (8,100 ha). A subsidence of 9 ft (2.75 m) at the San Jacinto Battleground State Park have resulted in a loss of 100 acres (405 ha) out of approx 430 acres (174 ha) of the original park property (6).

In 1961, Hurricane Carla produced maximum tides of 12 ft (3.6 m) on the gulfward side of Galveston Island, contrasted with 16-1/2-ft (5.03-m) tides on the upper reaches of Galveston Bay. Carla made landfall over 100 miles (161 km) southwest of Galveston, and a similar storm crossing land at the mouth of Galveston Bay would produce tides in excess of 22 ft (6.7 m). Over a quarter of a million people reside between the 600-mile (966-km) shoreline of Galveston Bay and the 20-ft (6.1 m) topographic contour (6).

Brownwood Subdivision of Baytown, a bayfront community consisting of 450 homes, has been lost to subsidence which has been in excess of 8 ft (2.4 m). Following a Corps of Engineers feasibility study for the construction of a protective levee, it was concluded that relocation of the residents would be more feasible than to undertake an expensive construction project. Revised estimates place subsidence costs attributable to this project in excess of $40,000,000.

In 1975, the results of an attempt to quantify the costs of subsidence in three heavily impacted areas comprising 945 sq miles (2,450 km^2) of Harris County were published. The study estimated annual costs in property value losses conservatively at $31,700,000/yr for the area between 1969 and 1973. The study concluded that it would be less expensive to import surface water than to continue to pump excessive amounts of groundwater (5).

Harris-Galveston Coastal Subsidence District

The Harris-Galveston Coastal Subsidence District, created to regulate the withdrawal of ground water in Harris and Galveston Counties to end subsidence, came into existence with the signing of House Bill 552 by the Governor of the state of Texas on May 23, 1975. On July 31, 1975, the district began to function when its first 15-member Board of Directors, appointed by local

government officials from Harris and Galveston Counties, were administered the oath of office.

The act creating the district provides that water wells located within the district, with a casing diameter in excess of 5 in. (130 mm) are required to have a permit to withdraw a specified amount of water for a period of not less than one year nor more than 5 yr. In addition, any well owner who has more than one well located within the district, regardless of the size of the well, must apply for a permit for each well. In reviewing permit applications, the district must consider the purpose of the act, the district's plan to end subsidence, the quantity and quality of surface water at competitive prices to ground water, the economic impact on the permit applicant, and the relative effect of pumping on subsidence. The authority to regulate ground-water withdrawal enables the district to control total ground-water withdrawals. During 1976, 1,613 permits were issued to over 600 well owners, permitting withdrawals in excess of 185 billion gal (0.69 Gm^3). Approximately one-third of the initial permits issued contained special provisions which required well owners to conduct feasibility studies relative to the use of surface water as alternative to ground water. All such well owners were required to present reports to the board of directors and the reports have served as a basis for requiring the increased use of surface water. Currently, the district is annually permitting over 2,300 wells and almost 900 owners.

Surface Water

The areas reflecting the major pumpage decrease were included in an area defined by the District Board of Directors in 1976 on adopting the first district plan for controlling subsidence. This area was called the "area of concentrated emphasis" and is shown in Fig. 6. The intent of the plan was to define the board's intentions to restrict ground-water development and pumpage in the area which had incurred the greatest amount of subsidence, was of low elevation, and which had surface water generally available. As shown in Fig. 2, the district has been effective in reducing groundwater pumpage in the area of concentrated emphasis. Between 1976 and 1978, total pumpage decreased from 449.8 mgd (19.48 m^3/s)–421.3 mgd (18.24 m^3/s), a decrease of 6.3%. The change in pumpage for the area of concentrated emphasis (Pasadena, Baytown-La Porte, NASA, Texas City, and Alta Loma), however, was from 184.5 mgd (8.084 m^3/s) in 1976 to 119.7 mgd (5.245 m^3/s) in 1978, a decrease of about 65%.

Two major developments in the area surface water system were completed since the creation of the district and have enabled the district to effect pumpage reductions. The Coastal Industrial Water Authority (CIWA) was created by the state legislature in 1969 to finance and construct conveyance and distribution facilities to transport water from Lake Livingston to the industrial area along the Houston Ship Channel. In late 1976, industries in the Pasadena-Deer Park area, the heart of the subsidence problem, began to receive water from CIWA. Average use of Trinity River water during 1978 was 106 mgd (4.56 m^3/s) in Harris and Galveston Counties (Table 1). When completely developed, the CIWA system will have a capacity of 1,200 mgd (51.96 m^3/s).

As the CIWA system was being developed, there was a corresponding increase in the use of surface water from Lake Houston. In the Fall of 1976, the city

of Houston completed the expansion of its water treatment plant from 50 mgd (2.2 m^3/s)–125 mgd (5.41 m^3/s). The net effect of the increased surface water use from Lake Houston showed an increase from about 145 mgd (6.28 m^3/s) for 1972–1973 to about 193 mgd (8.36 m^3/s) for 1978. Increased use of surface water in the city of Houston municipal system, made possible by the East Plant expansion in 1976, has contributed to subsidence control and allowed the city to shift more pumpage to wells in the western part of the city.

Total water production in the municipal system increased from 258 mgd (11.3 m^3/s) in 1976 to about 341 mgd (14.9 m^3/s) in 1978, or about 30%. Average groundwater use increased by 18.8%, from 165 mgd (7.14 m^3/s)–196 mgd (8.49 m^3/s), while average surface water use increased by almost 60%, from 93 mgd (4.1 m^3/s)–145 mgd (6.28 m^3/s). In 1976, ground water represented about 64%

TABLE 1.—Surface Water Yields and Use, Harris and Galveston Counties, by Source for 1978, in million gallons per day[a]

Source (1)	City of Houston (2)	Galveston County Water Authority (3)	San Jacinto River Authority (4)	Total (5)
		(*a*) Yields		
San Jacinto River	153		75	228
Brazos River		114		114
Trinity River	806[b]			806
Total	959	114	75	1,148
		(*b*) Use		
San Jacinto River	193		42	235
Brazos River		55		55
Trinity River	130[c]			130
Total	323	55	42	420

[a]1 mgd = 0.043 m^3/s.
[b]Does not include yield which might be realized by completion of the Wallisville Project.
[c]Includes 22.6 mgd from Southern Canal System which is used in Chambers County.

of the water used in the municipal system. In 1978, the portion was reduced to about 57%.

Unfortunately, it does not appear that the immediate future of surface water development will offer the opportunities for ground-water pumpage reductions which have been available in the recent past. The Houston area is outgrowing its water system at an alarming rate. The Houston-Galveston region cannot continue its rapid rate of growth unless the development of the existing surface water supply is accelerated. While a supply of surface water is available, its use awaits the development of treatment and distribution facilities.

There are three river systems which supply surface water to the area: the San Jacinto, the Brazos, and the Trinity. The San Jacinto system consists of Lake Conroe and Lake Houston; their yield is used by the city of Houston and the San Jacinto River Authority. The Brazos River is used as a source of surface water by industries in Texas City and will be used by Texas City

and La Marque when a 16-mgd (0.69-m^3/s) plant is completed in 1980. Also a 6-mgd (0.3-m^3/s) plant will be constructed by the Galveston County Water Authority to serve the areas east of League City and north of Dickinson Bayou. The Trinity River system consists of Lake Livingston, the Southern Canal system, and the CIWA conveyance facilities. The CIWA system is designed to ultimately deliver 1.2 billion gal/day (5.2 m^3/s) of water to the Lynchburg area on the Houston Ship Channel.

As shown in Table 1, there are over 1.1 billion gal/day (48 m^3/s) of surface water available from the three systems. The problem lies in getting the yield of the reservoirs to customers in a usable form. Current use of San Jacinto water is about 22 mgd (0.95 m^3/s); 28 mgd (1.2 m^3/s) in excess of the critical yield. Brazos water will be near its maximum potential use with the completion of the Texas City and the Galveston County Water Authority treatment plants. There are almost 700 mgd (30.7 m^3/s) available from Lake Livingston which are not being utilized today.

Areawide demand for water has grown to a point where tomorrow's estimates are today's realities. Only recently, the Trinity water supply was estimated to serve the Houston area until the next century. Unfortunately, population of water demand projections could not foresee the growth which has been experienced in recent years. As late as 1968, Houston municipal water demand was estimated to be 271 mgd (11.8 m^3/s) in 1980 (7). In 1978, the city used an average over 313 mgd (13.7 m^3/s) of potable water in its municipal system. Current projections indicate that all supplies of water currently available will be exhausted during the 1900s.

The immediate problem lies in getting the surplus water available from the Trinity River to the customer—building costly treatment plants and distribution facilities. In February of 1978, the City of Houston announced a 7-yr, $750,000,000 capital improvement program for the water system. Included are plans to increase groundwater production at existing well fields by about 30 mgd, to develop well fields outside of Harris County with a total pumpage of about 120 mgd, and to expand surface water treatment facilities by 200 mgd (5.20 m^3/s). The time required to develop the surface water system [5-1/2 yr for a 50 mgd (2.2 m^3/s)–100 mgd (4.33 m^3/s) surface water treatment plant] will pose serious problems for the water supply system and severely limit the Subsidence District's ability to curtail ground-water pumpage.

Conclusion

The Subsidence District has addressed several major problems in establishing itself as a regulatory agency during its brief history. Major problems are being addressed at the present which will determine if the approach to water resource management, as found to exist in Harris and Galveston Counties, will be workable.

Each step in the development of district rules, policies and procedures has been an original, if not unique, effort. The act creating the district is unique in Texas law and there were few usable procedures which had been established in other parts of the state. There are no other agencies regulating the production of ground water in Texas, although underground water conservation districts are empowered to do so. Compounding the problem of having to develop a regulatory process was a lack of ground-water use data. The district is currently

involved in the second phase of a comprehensive water management study. The first phase of the study was completed in late 1979 and includes the assimilation and evaluation of existing data relative to all aspects of the subsidence problem. The second phase will produce estimated subsidence conditions and damages which may be incurred under various groundwater pumpage conditions and the amount of groundwater pumpage which may be allowed in various sectors of the district.

While the cooperation of permit holders has been a major factor in the early success of the district, regulatory measures have not gone without resistance. In November 1975, the infant agency was sued by two groups of rice farmers which challenged the constitutionality of the creation of the district and the reasonableness of district permit fees (1). The action was taken following a public hearing which established a fee of $12.50/million gal permitted pumpage regardless of the type of water use of the location of regulated wells. The current permit fee rate is $4.50/million gal. The case was finally settled in Spring of 1978 when the Supreme Court of Texas, in denying plaintiff's petition for writ of error, upheld the district on all points raised by the suit.

Late in 1978, the Supreme Court further reinforced the legislature's authority to regulate groundwater for the purpose of controlling subsidence in *Friendswood Development Co., et al., Versus Smith—Southwest Industries, Inc., et al* (2). Although the district was not party to nor involved in the case, the court referred to the creation of the district and other legislative acts in establishing the intent of the legislature to limit the common law rule of absolute ownership of ground water. The court held that ground-water users were not liable for subsidence damages caused by past actions, but could be held responsible for damages due to future pumpage if such were conducted in a negligent or malicious manner.

Attempts to limit the authority or jurisdiction of the district have been made before the legislature in both sessions since the district was created. In 1977, agricultural interests sought total exemption from the act. The outcome of a very vigorous legislative effort was House Bill 390, which gave a 30% discount on permit fees paid for water pumped for irrigation of crops.

If the Subsidence District is to continue toward its mandate of controlling subsidence, several basic questions need to be addressed. Can surface water development keep pace with anticipated growth in area water demand? It appears that significant developments will not materialize sooner than 1985–1986, when two city of Houston treatment plants are tentatively scheduled for completion. The current total water demand is estimated to be in excess of 800 mgd (35.1 m^3/s) for the two counties. Maintaining ground-water pumpage at or near present rates will be difficult unless surface water development is accelerated or the growth in demand is curtailed.

Can broad-based conservation practices be instituted which will slow the growth in demand? The district is only beginning to assess opportunities for implementing conservation techniques. Examples can be cited of specific cases where wasteful practices could be eliminated and meaningful gains realized in a relatively short period of time. Yet a case could be made that conservation measures affected by industrial pollution control measures since the mid-sixties have "saturated the conservation market." Realization of water savings by individuals is a slow and continuing educational process worthy of attention, but not an area which will yield immediate water savings. About two-thirds of the ground water produced

in the two counties is consumed by the customers of some 240 cities, water districts, and other public utilities.

How much additional ground-water development can be permitted inside the district? Can ground-water be economically imported? Estimates by various sources of the "safe yield" of the aquifers have ranged from less than 490 mgd (21.5 m^3/s)–700 mgd (30.7 m^3/s). Obviously, estimates of total ground-water pumpage without subsidence or with a tolerable degree of subsidence would depend on assumptions relative to well distribution.

The Comprehensive Water Management Study previously referred to should provide a basis for dealing with such questions. It will represent the first conjunctive water use planning effort conducted for the entire two-county area. Implementing the recommendations of the plan, however, will require the cooperation of a very diverse and fragmented water community. The Subsidence District has no authority to build facilities, is prohibited from selling water, and has only the leverage of restricting ground-water pumpage to influence water development.

If the surface water delivery system is not expanded or growth in water demand is not curtailed, then subsidence will continue as an expense of growth; or, growth will be slowed as an expense of subsidence. There are undesirable aspects to each alternative.

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INDUCED SINKHOLES: AN ENGINEERING PROBLEM[a]

By John G. Newton[1]

INTRODUCTION

Recent sinkholes formed by collapses in the land surface have resulted in a variety of problems related to the maintenance and safety of man's structures and the pollution of existing and potential water supplies. These sinkholes, in relation to their occurrence, can be separated into two categories defined as "induced" and "natural." Induced sinkholes are those that can be related to man's activities, whereas natural ones cannot.

Induced sinkhole activity described here is restricted to that in Alabama; however, similar activity in the United States has occurred in Georgia, Maryland, Missouri, Pennsylvania, South Carolina, and Tennessee. An inventory to determine the extent of this problem has not been made even though damage resulting from it has been significant. In Alabama alone, costly damage and numerous accidents have occurred or nearly occurred as a result of collapses beneath highways, streets, railroads, buildings, sewers, gas pipelines, vehicles, animals, and people.

The purposes of this paper are to: (1) Describe forces involved in the development of induced sinkholes to aid in construction in problem areas or in repairing damage resulting from their occurrence; and (2) provide information that will identify or aid in the recognition of active and potential areas of sinkhole development that will allow planners to avoid them. Because of interest in this problem by engineering and planning groups, this paper was prepared for presentation at annual meetings of ASCE, held at Atlanta, Ga., 1979, and the Applied Geography Conference, held at Kent State University, Kent, Ohio, 1980. Information included here is taken largely from several reports by the writer during the period 1971–1976 (8,9,10).

[a]Presented at the October 22–26, 1979, ASCE, Annual Convention and Exposition, held at Atlanta, Ga.

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0175/$01.00.

Geologic and Hydrologic Setting

The terrane used to illustrate sinkhole development is a youthful basin underlain by carbonate rocks such as limestone and dolomite (Fig. 1). The basin contains a perennial or near-perennial stream. This particular terrane is used because it is very similar to that of 10 active areas of sinkhole development in Alabama that have been examined. Factors related to the development of sinkholes in these areas are generally applicable to other similar carbonate terranes. The terrane differs from those examined only in the inclination of beds, which is shown as horizontal for ease of illustration.

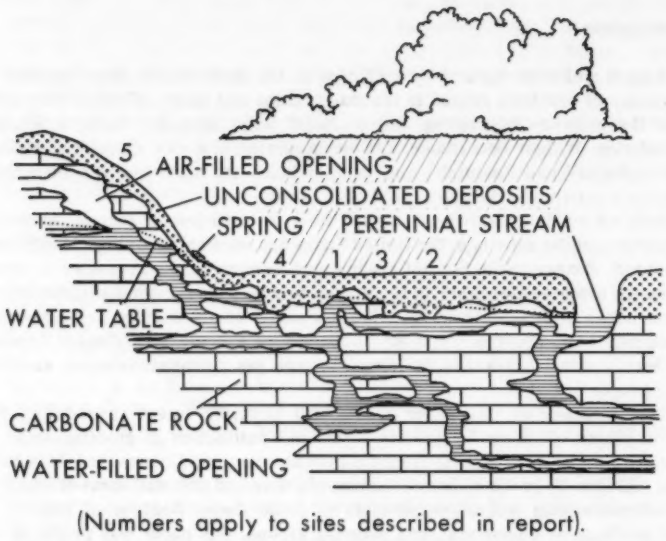

FIG. 1.—Schematic Cross-Sectional Diagram of Basin Showing Geologic and Hydrologic Conditions

The development of sinkholes is primarily dependent on past and present relationships between carbonate rocks and water, climatic conditions, vegetation, and topography, and on the presence or absence of residual or other unconsolidated deposits overlying bedrock. The source of water associated with the development of sinkholes is precipitation which, in Alabama, generally exceeds 1,270 mm annually. Part of the water runs off directly into streams, part replenishes soil moisture but is returned to the atmosphere by evaporation and transpiration, and the remainder percolates downward below the soil zone to ground-water reservoirs.

Water is stored in and moves through interconnected openings in carbonate rocks. Most of the openings were created, or existing openings along bedding

planes, joints, fractures, and faults were enlarged by the solvent action of slightly acidic water coming in contact with the rocks. Water in the interconnected openings moves in response to gravity from higher to lower altitudes, generally toward a stream channel where it discharges and becomes a part of the streamflow.

Water in openings in carbonate rocks occurs under both water-table and artesian conditions; however, this study is concerned primarily with that occurring under water-table conditions. The water table is the unconfined upper surface of a zone in which all openings are filled with water. The configuration of the water table conforms somewhat to that of the overlying topography but is influenced by geologic structure, withdrawal of water, and variations in rainfall. The lowest altitude of the water level in a drainage basin containing a perennial stream occurs where the water level intersects the stream channel (Fig. 1). Openings in bedrock underlying lower parts of the basin are water-filled. This condition is maintained by recharge from precipitation in the basin. Openings in bedrock between the land surface and the underlying water table in highland areas are air-filled (Fig. 1).

The general movement of water through openings in bedrock underlying the basin, even though the route may be circuitous, is toward the stream channel and downstream under a gentle gradient approximating that of the stream. Some water moving from higher to lower altitudes is discharged through springs along flanks of the basin because of the intersection of the land surface and the water table. The velocity of movement of water in openings underlying most of the lowland area is probably sluggish when compared to that in openings at higher altitudes.

A mantle of unconsolidated deposits consisting chiefly of residual clay (residuum), that has resulted from the solution of the underlying carbonate rocks, generally covers most of the bedrock in the typical basin described. Alluvial or other unconsolidated deposits often overlie the residual clay. The residuum commonly contains varying amounts of chert debris that are insoluble remnants of the underlying bedrock. Some unconsolidated deposits are carried by water into openings in bedrock. These deposits commonly fill solutionally enlarged joints, fractures, or other openings underlying the lowland areas. The buried contact between the residuum and the underlying bedrock, because of differential solution, can be highly irregular (Fig. 1).

Induced Sinkholes

It is estimated that more than 4,000 induced sinkholes, areas of subsidence, or other related features in this category have occurred in Alabama since 1900; most have occurred since 1950. During the same period, less than 50 new natural sinkholes have been reported. Induced sinkholes are divided into two types: those related to a decline in the water table, and those related to construction.

The following text is devoted to the description of the initial stage of development of induced sinkholes. Recent collapses in Alabama generally range from 1 m–90 m in diam and from 0.3 m–30 m in depth. Because of the rapidity of development, this type sinkhole is a significant hazard. This is particularly true where they are a threat to major transportation arteries (Fig. 2).

Decline of Water Table

A relationship between the formation of sinkholes and high pumpage of water

FIG. 2.—Collapse beneath Interstate Highway 59 in Birmingham, Ala.

from new wells was recognized in Alabama as early as 1933 (7). Subsequent studies in Alabama (8–12) have verified this relationship.

Dewatering or the continuous withdrawal of large quantities of water from carbonate rocks by wells, quarries, and mines in numerous areas in Alabama is associated with extremely active sinkhole development. Numerous collapses in these areas contrast sharply with their lack of occurrence in adjacent geologically and hydrologically similar areas where withdrawals of water are minimal. For example, in five active sinkhole areas examined by the writer, an estimated 1,700 collapses, areas of subsidence, or other associated features have formed in a total combined area of about 36 km^2. This phenomenon is not unusual; the relationship of this type of sinkhole occurrence to cones of depression created by water withdrawals in Pennsylvania and Africa has been well established by Foose (3,4).

Cause and Development

Two areas in Alabama in which intensive sinkhole development has occurred and is occurring have been studied in detail. Both areas were made prone to the development of sinkholes by major declines of the water table due to the withdrawals of ground water. The formation of sinkholes in both areas resulted from the creation and collapse of cavities in unconsolidated deposits caused by the declines (9,10).

Cavities in unconsolidated deposits overlying carbonate rocks in areas where there have been water-table declines have also been described and explored in Africa and Pennsylvania (2,6,4). The growth of one such cavity in Alabama

has been photographed through a small adjoining opening (8).

Excellent reports prior to 1971 associated the development of sinkholes and subsidence with subsurface erosion caused by pumpage, the position of the water table, or a lowering of the water table due to withdrawals of ground water. Robinson and others (12) attributed the development of sinkholes in a cone of depression to the increased velocity of ground water which caused the collapse of clay and rockfilled cavities in bedrock. Foose (3) associated the occurrence of recent sinkhole activity with pumping and a subsequent decline in the water table. He determined that their formation was confined to areas where a drastic lowering of the water table had occurred, that their occurrence ceased when the water table recovered, and that the shape of recent collapses indicated a lowering of the water table and withdrawal of its support. Jennings and others (6) associated development of sinkholes with pumpage and creation of cones of depression. They determined that sinkhole and subsidence problems increased where the water table was lowered and described the formation, enlargement, and collapse of cavities in unconsolidated deposits due to their downward migration.

Previous reports have described only indirectly or in part the hydrologic forces resulting from a decline in the water table that create or accelerate the growth of cavities that collapse and form sinkholes. These forces, based on studies in Alabama (9,10), are: (1) A loss of support to roofs of cavities in bedrock previously filled with water and to residual clay or other unconsolidated deposits overlying openings in bedrock; (2) an increase in the velocity of movement of ground water; (3) an increase in the amplitude of water-table fluctuations; and (4) the movement of water from the land surface to openings in underlying bedrock where recharge had previously been rejected because the openings were water-filled.

The same forces creating cavities and subsequent collapses also result in subsidence. The movement of unconsolidated deposits into bedrock where the strength of the overlying material is not sufficient to maintain a cavity roof, will result in subsidence at the surface (2).

To demonstrate forces that result in the development of cavities and their eventual collapse, a schematic diagram is shown in Fig. 3 that demonstrates changes in natural geologic and hydrologic conditions previously described and shown in Fig. 1. A description of the forces triggered by a lowering of the water table follows.

The loss of buoyant support following a decline in the water table can result in an immediate collapse of the roofs of openings in bedrock or can cause a downward migration of unconsolidated deposits overlying openings in bedrock. The buoyant support exerted by water on a solid (and hypothetically) unsaturated clay overlying an opening in bedrock for instance, would be equal to about 40% of its weight. This determination is based on the specific gravities of the constituents involved. Site 1 on Fig. 1 shows the unconsolidated deposits overlying a water-filled opening in bedrock. Site 1 on Fig. 3 shows the decline in the water table and the resulting cavity in the deposit formed by the downward migration of the unconsolidated deposit caused by the loss of support. The cavity may remain stable or it may enlarge upward by the spalling of the overlying deposit until the roof collapses.

The creation of a cone of depression in an area of water withdrawal results

in an increased hydraulic gradient (slope of the water table) toward the point of discharge (Fig. 3) and a corresponding increase in the velocity of movement of water. This force can result in the flushing out of the finer grained unconsolidated sediments that have accumulated in the interconnected solutionally enlarged openings. This movement also transports unconsolidated deposits migrating downward into bedrock openings to the point of discharge or to a point of storage in openings at lower altitudes.

The increase in the velocity of ground-water movement also plays an important role in the development of cavities in unconsolidated deposits. Erosion caused by the movement of water through unobstructed openings and against joints, fractures, faults, or other openings filled with clay or other unconsolidated

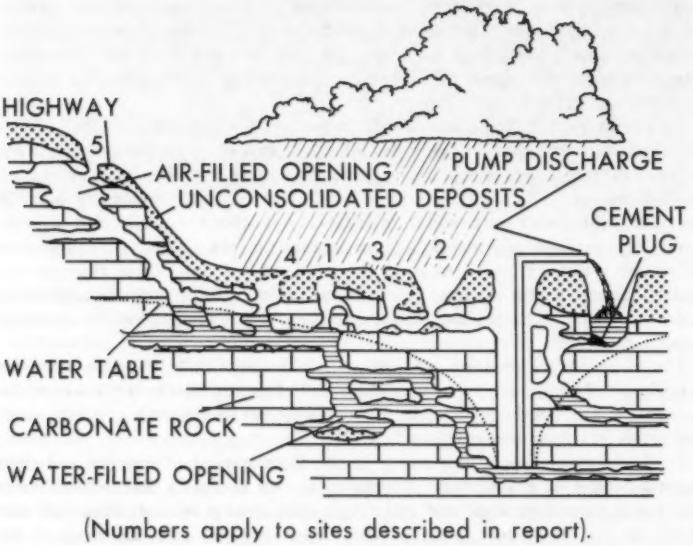

FIG. 3.—Schematic Cross-Sectional Diagram of Basin Showing Changes in Geologic and Hydrologic Conditions Resulting from Water Withdrawal

sediments results in the creation of cavities that enlarge and eventually collapse (7,12).

Pumpage results in fluctuations in ground-water levels that are of greater magnitude than those occurring under natural conditions. The magnitude of these fluctuations depends principally on variations in water withdrawal and on fluctuations in natural recharge (precipitation). The repeated movement of water through openings in bedrock against overlying residuum or other unconsolidated sediments causes a repeated addition and subtraction of support to the sediments and repeated saturation and drying. This process might be best termed "erosion from below" because it results in the creation of cavities in unconsoli-

dated deposits, their enlargement, and eventual collapse. Fluctuations of the water table against the roof of a cavity in unconsolidated deposits have been observed and photographed through a small collapse in the center of the roof. These fluctuations, in conjunction with the movement of surface water into openings in the ground, resulted in the formation of the cavity and its collapse (10).

A drastic decline of the water table in a lowland area (Fig. 3) in which all openings in the underlying carbonate rock were previously water-filled (Fig. 1) commonly results in induced recharge of surface water. This recharge was partly rejected prior to the decline because the underlying openings were water-filled. The quantity of surface water available as recharge to such an area is generally large because of the runoff moving to and through it from areas at higher altitudes.

The inducement of surface water infiltration through openings in unconsolidated deposits interconnected with openings in underlying bedrock results in the creation of cavities where the material overlying the openings in bedrock is eroded to lower altitudes. Repeated rains result in the progressive enlargement of this type cavity. A corresponding thinning of the cavity roof due to this enlargement eventually results in a collapse. The position of the water table below unconsolidated deposits and openings in bedrock that is favorable to induced recharge is shown in Fig. 3. Sites 2, 3, and 4 on Fig. 3 demonstrate a collapse and cavities in unconsolidated deposits that were formed primarily or in part by induced recharge. The creation and eventual collapse of cavities in unconsolidated deposits by induced recharge is the same process described by many authors as "piping" or "subsurface mechanical erosion" where it has been applied mainly to collapses occurring on noncarbonate rocks (1).

In an area of sinkhole development where a cone of depression is maintained by constant pumpage (Fig. 3), all the forces described are in operation even though only one may be principally responsible for the creation of a cavity and its collapse. For instance, the inducement of recharge from the surface (site 2 on Fig. 3) where the water table is maintained at depths well below the base of unconsolidated deposits, can be solely responsible for the development of cavities and their collapse. In contrast, a cavity resulting from a loss of support (site 1 on Fig. 3) can be enlarged and collapsed by induced recharge if it has intersected openings interconnected with the surface. In an area near the outer margin of the cone (site 4 on Fig. 3), the creation of a cavity and its collapse can result from all forces. The cavity can: (1) Originate from a loss of support; (2) be enlarged by the continual addition and subtraction of support and the alternate wetting and drying resulting from water-level fluctuations; (3) be enlarged by the increased velocity of movement of water; and (4) be enlarged and collapsed by water induced from the surface.

Construction

Collapses resulting from construction are far less numerous than those due to a decline in the water table; however, they have resulted in extensive damage in Alabama. in this paper, the term "construction" applies not only to the erection of a structure, but also to any diversion of natural drainage and includes the clearing of timber in rural areas.

Cause and Development

The simplest cause of sinkholes or subsidence that can result from construction is loading. The emplacement of weight alone on unconsolidated deposits can result in compaction. The compaction can be irregular where the deposits overlie an uneven bedrock surface or openings in bedrock. Differential compaction and accompanying subsidence under these circumstances can result in foundation problems. The presence of natural or induced cavities in bedrock or unconsolidated deposits can result in a collapse when the overlying roof is subjected to loading.

Construction on unconsolidated deposits that overlie air-filled openings in bedrock (site 5 on Fig. 1) can result in the formation of a sinkhole (site 5 on Fig. 3). In construction, grading and the removal of trees creates new openings that connect the land surface with openings in the underlying bedrock. The concentration of surface runoff in drains or impoundments increases the downward movement of water. This downward movement sometimes erodes and transports unconsolidated deposits into underlying openings in bedrock forming a cavity in the deposits that eventually enlarges and collapses. This process is the same as that described under induced recharge (piping) where the water table has been lowered by pumping. An excellent example of a collapse caused by a diversion of drainage and resulting "piping," including photographs of an uncollapsed tunnel (pipe) and cavity in unconsolidated deposits overlying carbonate rocks, has been described by the writer (8). The piping process has also been described and illustrated in a carbonate terrane in Alabama where collapses have resulted in retention-basin failures (13).

The "piping" action can also apply to the development of sinkholes where water has been impounded on unconsolidated deposits that overlie carbonate rocks containing water-filled openings. On the floor of the impoundment, water moving through openings in the unconsolidated deposits to openings in carbonate rocks can form and collapse cavities in the deposits. This process generally occurs where there is considerable head or pressure exerted by the impounded water and where openings in the carbonate rocks have a discharge point outside the impoundment at a lower altitude. The increase in the velocity of impounded water moving through unconsolidated deposits to underlying openings in bedrock will have an erosive capacity similar to that described previously in a cone of depression caused by pumpage. The saturation of previously unsaturated unconsolidated deposits by the impoundment will also enhance the downward transport of the deposits. This action is probably responsible for the formation of numerous sinkholes in the impoundment behind one of Alabama's largest dams (8).

The most damaging collapse due to construction in Alabama involved Interstate Highway 59 near Attalla. A collapse about 3 m in diam located in a drainage ditch along the highway allowed surface drainage to enter the ground. Based on a dye test, it was determined that the drainage discharged at a lower altitude beneath the fill of a lower lane. The lubrication of residual clay underlying the lane by this discharge and by some additional water from an unidentified source, resulted in a landslide and subsequent highway failure.

Problem Recognition

Early recognition of sinkhole or potential sinkhole problems can result in

minimizing or avoiding future damage and safety problems. Where induced sinkholes cannot be avoided because of their density or because construction or planning is in advanced stages, additional engineering can be incorporated to minimize their effect. Bridging, adding additional support, the removal of unconsolidated deposits overlying bedrock, grouting, minimizing the diversion of natural drainage, and the construction of flumes and other impermeable drainage systems to remove surface water from problem areas are some of the means that can be incorporated to aid in accomplishing this objective.

Induced sinkholes can generally be differentiated from natural ones by their physical characteristics. Their walls generally are vertical or steeply inclined and consist mostly of unconsolidated deposits that support little or no vegetation. Their rims are relatively sharp and show few signs of long-term erosion. Where floors of these sinkholes consist of unconsolidated deposits, fractures in them indicating recent downward movement are commonly present. These sinkholes are generally associated with other similar collapses and with nearby wells, deep recessed quarries, or mines that pump large quantities of ground water. Most important, their occurrence or potential occurrence is most common in the geologic and hydrologic setting described (Fig. 3). Induced sinkholes are similar to very recent natural collapses; however, the latter would generally be rare, would be associated with older natural sinkholes, and would not necessarily be associated with nearby pumpage.

In general, induced sinkholes or subsidence resulting from significant declines in the water table pose a more serious problem than those resulting from natural processes. Their occurrence can be numerous and their appearance sudden. It is estimated, based on available information, that about 40% of these collapses have a major surface dimension exceeding 5 m; about 10% have a major dimension exceeding 9 m; and about 3% have a major dimension exceeding 15 m.

Many induced sinkholes develop with little or no advance warning; others are preceded by the formation of recognizable features at the land surface. Most of the same features also indicate or would be expected to indicate the development of some natural sinkholes. Unfortunately, several of the features can also be attributed to other common causes. The recognition of several of the features is sometimes necessary for positive identification. These features [Features 1, 2, 3, 5, 6, 7, and 8 are described by the writer and Hyde (9)] are as follows:

1. Circular and linear cracks or fractures in soil, asphalt, and concrete paving and floors.
2. Appearance of depressions in soil or pavement that commonly result in the ponding of water.
3. Slumping, sagging, tilting, or warping of highways, rails, fences, curbings, pipes, poles, sign boards, and other vertical or horizontal structures.
4. The downward movement of small-diameter vertical structures such as poles or posts.
5. Fractures in foundations and walls that are sometimes accompanied by jammed doors and windows. Loud, sharp cracking noises in or under a structure often accompanies damage to foundations.
6. Small conical holes that appear in the ground in a relatively short period of time.

7. Diversion of drainage into small holes or fractures in the ground without rapid filling.

8. Sudden muddying of water in a well that has been producing clear water. This indication often is associated with precipitation.

9. Water spouts erupting during rainstorms from openings in soil in basins adjacent to highland areas.

10. Relatively small openings in the soil zone that are sometimes angular in shape and appear to be deep.

11. Discolored or dead vegetation in slight depressions.

12. Tilted or toppled trees with depressions at base.

13. Scour holes in areas subject to flooding where water erodes soil into the subsurface. Similar openings without eroded surfaces in areas having a grass cover.

14. Stream discharging into ground or spring reentering ground. Where flow exceeds water loss, small whirlpools accompanied by sucking or draining noises are sometimes present.

15. Sudden draining of pond or inability of new pond to hold water.

16. Depressions in wooded areas where collapses have occurred but where root zone maintains soil and leaf cover.

17. Sound of running water in subsurface being emitted through small openings at land surface.

18. Continuous muddying of water at high pumping rates during development of new well.

Conclusions

Induced sinkholes are those related to man's activities, whereas natural sinkholes are not. Induced sinkholes are divided into two types: those resulting from a decline in the water table due to ground-water withdrawals and those resulting from construction. Almost all induced sinkholes occur where cavities develop in residual or other unconsolidated deposits overlying openings in carbonate rocks. The downward migration of the deposits into underlying openings in bedrock and the formation and collapse of resulting cavities are caused or accelerated by a decline in the water table that results in: (1) Loss of buoyant support; (2) increase in the velocity of movement of water; (3) water-level fluctuations at the base of unconsolidated deposits; and (4) induced recharge. Most sinkholes resulting from construction are due to the diversion of drainage over openings in bedrock.

Early recognition of sinkhole or potential sinkhole activity can result in minimizing or avoiding the problem. Many collapses develop with little or no advanced warning; others are preceded by recognizable features at the land surface.

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SUBDOMAIN INTEGRATION MODEL OF GROUND-WATER FLOW

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INTRODUCTION

Numerical solution of the governing differential equations describing one-dimensional ground-water flow has received substantial attention since the advent of modern computers. Generally, numerical models are developed which employ the well-known finite-difference or Galerkin finite element methods (1,2) to approximate the governing equations, resulting in a model which can be solved generally only with computers of at least the minicomputer class. Recently, the "method-of-lines" (3) was used to solve the nonlinear unconfined ground-water flow equations resulting in a numerical model which can be accommodated by a programmable hand-held calculator. The algorithm used a "shooting method" which required an iteration process to obtain the desired accuracy. The main purpose of this paper is to present another approach to solving nonlinear (and linear) problems such as ground-water flow processes which also may be accommodated by programmable calculators. The numerical approach used is the subdomain integration version of the weighted residual methods as applied to solving for spatial coordinates as a function of the ground-water (or piezometric) surface.

The objectives of this paper are threefold. The first objectives is to present the subdomain integration numerical method as applied to a specific class of one-dimensional transport problems. Hromadka and Guymon (6,7,8) developed this numerical modeling procedure in detail and compare modeling efficiency to the well known finite difference and Galerkin finite element methods, and conclude that the subdomain integration procedure leads to a more accurate numerical model for the various problems tested. Extension of the modeling method to one-dimensional and two-dimensional linear and nonlinear advection-diffusion problems are the subject of current papers (9,10,11).

The second objective of this paper is to determine a subdomain integration

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Note.—Discussion open until June 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on October 15, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0187/$01.00.

numerical model which solves for spatial coordinates rather than solving for the governing flow equation's state variable. This approach somewhat eliminates nonlinearity (due to state variable dependent parameters) because the nonlinear parameters are evaluated at a constant value of the state variable along the boundaries of each subdomain.

The third objective is to simplify the resulting subdomain integration numerical model into an approximation which can be accommodated by programmable calculators. For the specialized problems tested, the simplified subdomain integration approximation produced good results when compared to available analytic solutions.

Governing One-Dimensional Ground-Water Flow Equations

One dimensional, unsteady ground-water flow in a confined homogeneous aquifer of a nearly uniform thickness is generally described by a linear partial differential equation of the form

$$\alpha \frac{\partial^2 h}{\partial x^2} = \frac{\partial h}{\partial t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

subject to appropriate boundary and initial conditions. In Eq. 1, h = a convenient reference of piezometric head (Fig. 3); x and t = spatial and temporal coordinates; and α = the quotient of transmissivity T, and effective porosity, n. Examples of approximately one-dimensional ground-water flow include the movement of water between a stream and the aquifer in response to a change in stage, and aquifer recharge from streams, canals, and irrigation ditches due to a sudden increase in stage.

The nonlinear partial differential equation describing one-dimensional unconfined ground-water flow is the well known Boussinesq equation

$$\frac{\partial}{\partial x}\left(K_x h \frac{\partial h}{\partial x}\right) = n \frac{\partial h}{\partial t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which K_x = the saturated hydraulic conductivity; and h = the hydraulic head. Due to the nonlinearity of Eq. 2, only a few quasianalytical solutions exist for select problems (4,5).

Numerical Model

The subdomain integration method (6–10) is applied to the governing flow equation of unconfined (and confined) ground-water flow. The boundary and initial conditions are assumed defined such that the spatial coordinate x can be described as a function of h (Fig. 1). The h-axis domain Ω is discretized by n nodal points into n subdomains Ω_j such that

$$\Omega = \bigcup_{j=1}^{n} \Omega_j \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\text{in which} \quad \Omega_j \cap \Omega_{j+1} = \frac{1}{2}(h_j + h_{j+1}); \quad j = 1, 2, \ldots, n-1 \quad \ldots\ldots\ldots\ldots (4)$$

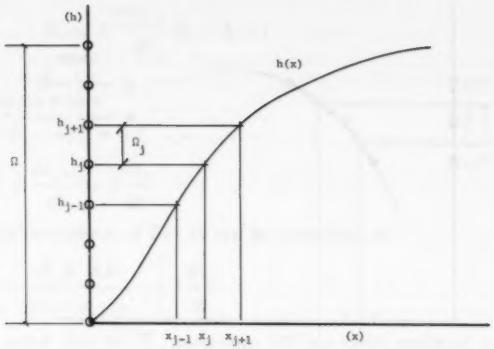

FIG. 1.—Nodal Point Distribution on h-Axis Global Domain Ω

For an interior subdomain Ω_j, $1 < j < n$, let

$$h' = \Omega_{j-1} \cap \Omega_j = \frac{1}{2}(h_{j-1} + h_j); \quad h'' = \Omega_j \cap \Omega_{j+1} = \frac{1}{2}(h_j + h_{j+1}) \quad \ldots \quad (5)$$

$$x' = x(h'); \quad x'' = x(h'') \quad \ldots \quad (6)$$

The integration of the nonlinear Eq. 2 over Ω_j is modeled by

$$\left(K_s h \frac{dh}{dx}\right)\Bigg|_{x''} - \left(K_s h \frac{dh}{dx}\right)\Bigg|_{x'} = n \frac{d}{dt} \int_{x'}^{x''} h\, dx + nh' \frac{dx'}{dt} - nh'' \frac{dx''}{dt} \quad \ldots \quad (7)$$

For a linear trial function approximation on subdomain Ω_j (Fig. 2)

$$\int_{x'}^{x''} h\, dx = \frac{1}{8}\left[(x_j - x_{j-1})(h_{j-1} + 3h_j) + (x_{j+1} - x_j)(h_{j+1} + 3h_j)\right] \quad \ldots \quad (8)$$

in which each h_i is constant and $x_i = x_i(t)$. Thus

$$\frac{d}{dt} \int_{x'}^{x''} h\, dx = \frac{1}{8}\left[(h_{j-1} + 3h_j)\left(\frac{dx_j}{dt} - \frac{dx_{j-1}}{dt}\right)\right.$$

$$\left. + (h_{j+1} + 3h_j)\left(\frac{dx_{j+1}}{dt} - \frac{dx_j}{dt}\right)\right] \quad \ldots \quad (9)$$

$$\frac{dx'}{dt} = \frac{1}{2}\left(\frac{dx_{j-1}}{dt} + \frac{dx_j}{dt}\right); \quad \frac{dx''}{dt} = \frac{1}{2}\left(\frac{dx_j}{dt} + \frac{dx_{j+1}}{dt}\right) \quad \ldots \quad (10)$$

Equation 9 can be rewritten as

$$\frac{d}{dt} \int_{x'}^{x''} h\, dx = \frac{dx_{j-1}}{dt}\left(-\frac{h_{j-1} + 3h_j}{8}\right) + \frac{dx_j}{dt}\left(\frac{h_{j-1} - h_{j+1}}{8}\right)$$

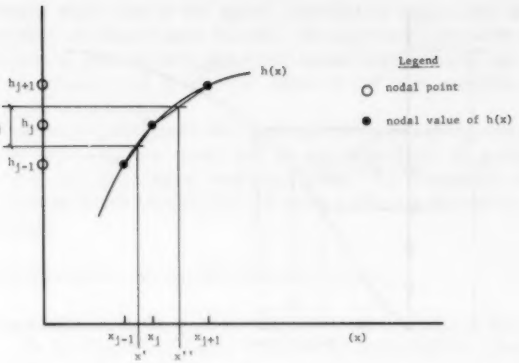

FIG. 2.—Linear Trial Function Approximation on Subdomain Ω_j

$$+ \frac{dx_{j+1}}{dt}\left(\frac{h_{j+1} + 3h_j}{8}\right) \quad \ldots \quad (11)$$

and $$h' \frac{dx'}{dt} - h'' \frac{dx''}{dt} = \frac{dx_{j-1}}{dt}\left(\frac{h_{j-1} + h_j}{4}\right) + \frac{dx_j}{dt}\left(\frac{h_{j-1} - h_{j+1}}{4}\right)$$

$$- \frac{dx_{j+1}}{dt}\left(\frac{h_j + h_{j+1}}{4}\right) \quad \ldots \quad (12)$$

From the preceding linear trial function approximations, Eq. 7 is rewritten as

$$\frac{d}{dt}\int_{x'}^{x''} h\,dx + h' \frac{dx'}{dt} - h'' \frac{dx''}{dt} = \frac{dx_{j-1}}{dt}\left(\frac{h_{j-1} - h_j}{8}\right)$$

$$+ \frac{dx_j}{dt}\left(\frac{3h_{j-1} - 3h_{j+1}}{8}\right) + \frac{dx_{j+1}}{dt}\left(\frac{h_j - h_{j+1}}{8}\right) \quad \ldots \quad (13)$$

For the linear trial function approximation for h on Ω_j

$$\left(K_s h \frac{\partial h}{\partial x}\right)\Bigg|_{x'} = K_s h' \frac{(h_j - h_{j-1})}{(x_j - x_{j-1})} \quad \ldots \quad (14)$$

$$\left(K_s h \frac{\partial h}{\partial x}\right)\Bigg|_{x''} = K_s h'' \frac{(h_{j+1} - h_j)}{(x_{j+1} - x_j)} \quad \ldots \quad (15)$$

in which $K_s h' (h_j - h_{j-1})$ and $K_s h'' (h_{j+1} - h_j)$ are constant for all time due to the numerical approximation solving for specified spatial coordinates x_j (Fig. 2). Combining Eqs. 7, 13, 14, and 15 gives the subdomain integration numerical model for Eq. 2 on Ω_j:

$$\frac{8\hat{K}_s h'' (h_{j+1} - h_j)}{(x_{j+1} - x_j)} - \frac{8\hat{K}_s h' (h_j - h_{j-1})}{(x_j - x_{j-1})} = \frac{dx_{j-1}}{dt}(h_{j-1} - h_j)$$

$$+ 3\frac{dx_j}{dt}(h_{j-1} - h_{j+1}) + \frac{dx_{j+1}}{dt}(h_j - h_{j+1}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

in which $\hat{K}_s = K_s / n$.

For problems where

$$h_{j+1} - h_j \simeq h_j - h_{j-1} = \Delta h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

and

$$\frac{dx_{j-1}}{dt} + \frac{dx_{j+1}}{dt} \simeq 2\frac{dx_j}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

the numerical statement of Eq. 16 can be simplified as

$$\frac{\hat{K}_s h'' \Delta h}{(x_{j+1} - x_j)} - \frac{\hat{K}_s h' \Delta h}{(x_j - x_{j-1})} = \Delta h \frac{dx_j}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

It may be noted that the K_s parameter has not been assumed constant in the numerical model derivations, i.e., due to the approach of solving for spatial coordinates x_j, nonlinear terms such as $hK(h)$ will be constant on the boundaries of Ω_j.

Equation 17 is integrated with respect to time to give the model approximation for Eq. 2 on Ω_j

$$\int_{t=k\Delta t}^{t=(k+1)\Delta t} \left[\frac{\hat{K}_s h''}{(x_{j+1} - x_j)} - \frac{\hat{K}_s h'}{(x_j - x_{j-1})} \right] dt + x_j(k\Delta t) = x_j[(k+1)\Delta t] \quad (20)$$

Integrating Eq. 20 over a small Δt timestep gives

$$\frac{\hat{K}_s h'' \Delta t}{(x_{j+1}^2 - x_j^2) - (x_{j+1}^1 - x_j^1)} \ln\left(\frac{x_{j+1}^2 - x_j^2}{x_{j+1}^1 - x_j^1}\right)$$

$$- \frac{\hat{K}_s h' \Delta t}{(x_j^2 - x_{j-1}^2) - (x_j^1 - x_{j-1}^1)} \ln\left(\frac{x_j^2 - x_{j-1}^2}{x_j^1 - x_{j-1}^1}\right) + x_j^1 = x_j^2 \quad \ldots\ldots\ldots\ldots \quad (21)$$

in which in Eq. 21 the superscripts 1 and 2 designate x-coordinate values at time $k\Delta t$ and $(k+1)\Delta t$, respectively. Equation 21 can be further simplified by letting

$$\frac{\hat{K}_s h'' \Delta t}{(\bar{x}_{j+1} - \bar{x}_j)} - \frac{\hat{K}_s h' \Delta t}{(\bar{x}_j - \bar{x}_{j-1})} + x_j^1 = x_j^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

in which it is assumed that (12)

$$\bar{x}_j = \frac{1}{2}[3x_j(t = k\Delta t) - x_j(t = k\Delta t - \Delta t) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

Therefore, an explicit formulation for the approximation of Eq. 2 is developed whereby each future x-coordinate, e.g., x_j^2, can be determined from previously determined data. The development of an equivalent numerical model for Eq. 1 follows analogously to the preceding derivation (6,7,8).

Model Applications

To demonstrate the accuracy of the proposed numerical method, the first

problem presented is an idealization of ground-water flow from a confined aquifer that forms the banks of a stream. It is assumed that drawdown is sufficiently small so that Eq. 1 describes the flow process. The initial and boundary conditions assumed in order to simulate an instantaneous step change in the piezometric profile of the aquifer are (Fig. 3)

$$h(x,0) = h_o; \quad h(\infty,t) = h_o, t \geq 0; \quad h(0,t) = 0, t > 0 \quad \ldots\ldots\ldots\ldots \quad (24)$$

The analytical solution to Eqs. 1 and 24 is given by

$$h = h_o \operatorname{erf}\left(\frac{x}{\sqrt{4\alpha t}}\right)$$

in which h_o = the assumed constant step change of stage. For example purposes, the substitutions of a unit step change in stage and $\alpha = 0.25$ (units of L^2/T) were used.

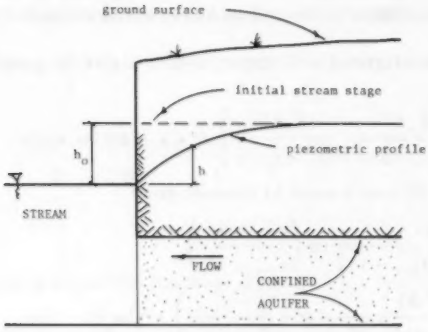

FIG. 3.—Flow from Confined Aquifer to Stream Due to Drop in Stage

Due to the several simplifying assumptions used to reduce the complexity of Eq. 16 to the approximation of Eq. 22, small timestep increments were required. The assumption of Eq. 17 is met by suitable discretization of Ω by equally spaced nodal points. The assumption of Eq. 18, however, limits the time rate of change of the state variable profile, i.e., in the initial portions of the problem solution when the piezometric profile changes relatively rapidly, much smaller timestep increments were required than when the piezometric profile varied more slowly. In order to keep the program size small, several simulations were made using various constant timestep sizes. The resulting modeled profiles were compared until negligible differences in the computed piezometric profiles were observed with decreasing timestep size. The resulting piezometric profile at various intervals of time are compared to model results in Fig. 4. From the figure, good results were obtained in the use of Eq. 22 to model the linear formulation of Eq. 1.

The second problem presented is the application of the model of Eq. 22 to solution of the nonlinear formulation of Eq. 2. The problem considered is

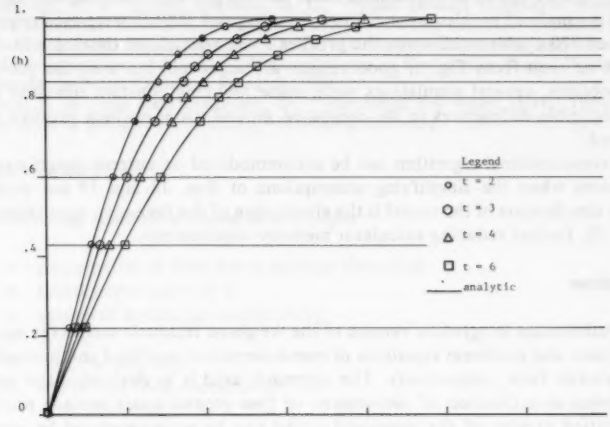

FIG. 4.—Model Results in Predicting Piezometric Profile

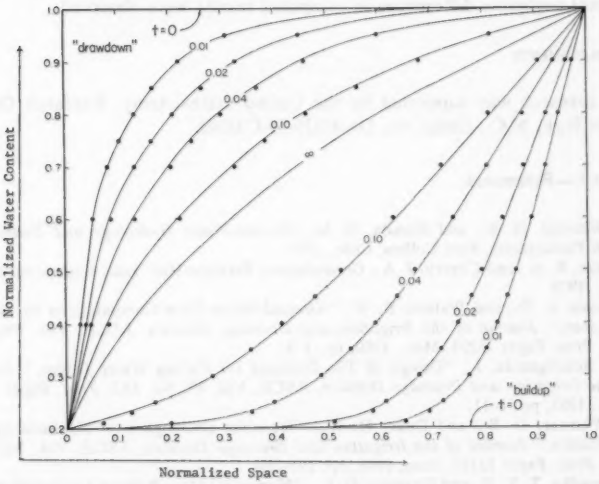

FIG. 5.—Model Results in Predicting Free Water Surface [Dots Represent Modeled Results; Solid Line Represents Analytic Solution to Equation 2 (3)]

the estimation of unconfined ground-water surface profiles for an instantaneous step change in elevation between two reservoirs separated by a given length of soil. Figure 5 shows dimensionless water surface profiles and computed results. For the normalized problems considered, time steps of .000125 were used requiring a total of 720 cycles to advance the profiles from normalized time of 0.01–0.10. As can be seen from Fig. 5, good results are achieved. As with the previous test problems, several simulations were made reducing constant timestep sizes until negligible differences in the computed ground-water surface profiles were observed.

The computational algorithm can be accommodated on current programmable calculators when the simplifying assumptions of Eqs. 18 and 19 are used. A further simplication of the model is the elimination of the time-step approximation of Eq. 23, further reducing calculator memory requirements.

Conclusions

The subdomain integration version of the weighted residuals method is applied to the linear and nonlinear equations of one-dimensional confined and unconfined ground-water flow, respectively. The approach used is to determine the spatial coordinates as a function of piezometric or free ground-water surface profiles. A simplified version of the numerical model can be accommodated by current programmable calculators. Good results were obtained when applying the proposed numerical model to the problems considered. However, several simulations were required for each problem tested, progressively reducing timestep sizes until negligible differences in computed results were observed.

Acknowledgments

This research was supported by the United States Army, Research Office, Triangle Park, N.C., Grant No. DAAG29-79-C-0080.

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Appendix II.—Notation

The following symbols are used in this paper:

h = piezometric or free water surface elevation;
h_j = nodal point value of h;
K_s = saturated hydraulic conductivity;
$\hat{K}_s$ = K_s/n;
n = specific porosity;
x, t = space and time coordinates;
erf = error function;
α = hydraulic diffusivity;
Δt = time step;
Ω = domain of definition; and
Ω_j = subdomain j.

TENSILE STRENGTH OF PVC AND ABS WELL CASING JOINTS

By Carl E. Kurt,[1] M. ASCE and W. Denney Pate[2]

INTRODUCTION

The use of thermoplastic materials as water well casings has become increasingly popular in the water well industry because of their favorable cost, ease in installation, and excellent performance. Well casings are generally long and must be made from sections of pipe, usually 10 ft (3 m) or 20 ft (6 m) in length. Therefore, between each section of casing, a structural joint is required. In a well casing, this joint must be capable of resisting a different type of load than the typical joint found in a water distribution or sewer system. While a joint in a well casing may experience some radial loads during and after installation, the primary load will be an axial one along the axis of the casing. Typical axial loads result from the dead weight of the casing or from the drilling machine. These loads may be of relatively short duration or of a longer duration depending upon installation procedures and casing design. After a review of the literature, no data were found to define a performance criteria of commercially available joint configurations subjected to axial tension loads. The primary objective of this paper is to present data generated from an experimental test program for thermoplastic casing joint systems subjected to short-term and long-term axial tension loads. The casings were made from two different thermoplastic materials, polyvinyl chloride (PVC) and acrylonitrile-butadiene-styrene (ABS). Based on the results from this study, a proposed design stress is presented for several joint configurations.

LITERATURE REVIEW

It has been well-documented that the strength of thermoplastic materials is a function of many parameters. For a given plastic formulation, temperature, chemical environment, rate of loading, loading history, and the extrusion process affect the structural performance of the thermoplastic part (2,6,9,11,13). Therefore, it is very important that the test conditions be carefully defined so the

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Note.—Discussion open until November 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on September 18, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0197/$01.00.

test results obtained can be properly interpreted.

Because commercially available materials were selected for the test program, full-scale sections of casing and joint configurations were tested. Gotham and Turner (5) conducted long-term tension tests on parallel-faced and waisted specimens cut from PVC sheet stock. The parallel-sided specimens continuously failed before the waisted specimens. Nesbitt (10) presented typical PVC stress regression data for PVC materials. These data were obtained from pressure tests conducted at room temperature.

Because well casings are made by an extrusion process, significant residual stresses are locked into the casing (3,14). Since one of the primary long-term loads for a casing is in the axial direction, the long-term strength in the circumferential direction may be different than the strength in the axial direction. For bell coupling joints, additional residual stresses are locked into the system due to the belling operation. Therefore, the motivation for full-scale specimen testing was reinforced.

Findley and Tracy (4) conducted significant long-time creep tests on unplasticized PVC specimens at constant stress levels. In this work, a prediction equation including the appropriate constants to describe the axial creep of small specimens was presented. Akay (1) expanded the application of the axial creep prediction equation for many polymers over a range of temperatures. In general, as the temperature increases, the axial creep of thermoplastics, such as ABS, at a given stress level will increase (6).

Test Program

Sections of commercially available PVC and ABS well casings between 4 in. (100 mm) and 8 in. (200 mm) in diameter were collected from three different PVC casing manufacturers and one ABS casing manufacturer.

The most common joint for PVC casing found in the water well industry is the bell coupling molded at one end of the casing. The joint is installed in the field using a primer and a solvent cement. The strength of this joint is a function of many important variables. They include: (1) Fit between the two components of the joint; (2) the use or nonuse of a primer, (3) the viscosity and chemical composition of the solvent cement; (4) the temperature and relative humidity of the environment while the joint is curing; and (5) the cure time before application of the load. If proper materials and procedures are used to make the joint, then a good structural joint should result.

Another PVC joint configuration tested was a Schedule 40 male/female threaded adapter system. The adapters were solvent cemented to the casing before installation and then adjacent casing sections were mechanically attached. The last PVC joint configuration investigated was a Schedule 40 slip coupling which was solvent cemented to adjacent sections of casing.

An ABS threaded casing was also evaluated. It consisted of male or female threads machined into the casing. A metal band was provided around the female end of the casing to protect the casing end. At the well site, adjacent sections of casing are threaded together and installed in the bore hole.

Each joint configuration was subjected to short-term and long-term tension loads. A study to evaluate the effect of cure environment and cure time on the short-term strength for solvent-cemented PVC bell couplings was conducted.

One series of 4-in. (100-mm) and 6-in. (150-mm) PVC bell couplings was cured in an 80° F (27° C) air environment whose average relative humidity was 65%. Another series of 6-in. (150-mm) PVC bell couplings was cured in the same air environment for 10 min and then placed in a water tank until tested. Cure time varied between 10 min and 24 h. After the appropriate cure time, the tension test fixtures were installed on the specimen and a short-term tension test was conducted.

For all PVC solvent-cemented joints, a primer and heavy duty solvent cement was selected based on the recommendation of a solvent-cement manufacturer. The procedures suggested by the manufacturer were followed when making all joints.

The primer was a colored, nonbodied, fast acting, water-thin primer system. A second coat of primer was applied to the fitting socket. The solvent cement was a gray, heavy-body, fast-curing, high-strength PVC solvent cement. It was

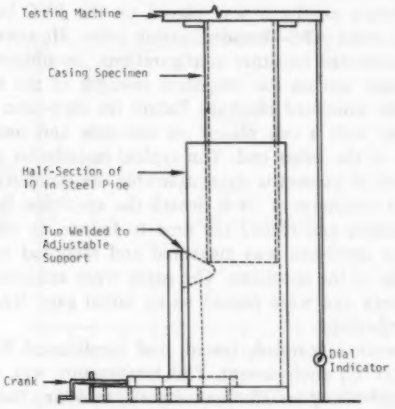

FIG. 1.—Simulated Blockage Test Fixture

formulated to cure somewhat slower than similar high viscosity cements. The Brookfield Viscosity was 1,600 cps minimum at 73° F (23° C), and the minimum wet film thickness was 0.028 in. (0.71 mm). Both primer and solvent cement are readily available on the commercial market. Since it was determined that the full joint strength was developed after 24 h of cure time, the PVC male/female adapters and slip coupling joints were solvent-cemented, air-cured, and tested after 24 h of cure time.

The ABS threaded casing evaluated during the program was relatively thick (SDR17) well casing. The threads were cut directly into the casing wall. The primary variable in the threaded casing test program was to determine the effect of torque used to tighten the joint on the joint axial tensile strength.

Some of the joint configurations were also subjected to a simulated blockage test. Details of this test and fixture are presented in Fig. 1 and Ref. 8. However, this test primarily consisted of pushing the casing and joints past a known

obstruction and measuring the peak axial compression force. After the test was completed, the joint integrity was observed.

All short-term tests were conducted on a 100 K Testing Machine at a head travel speed of 0.5 in./min (13 mm/min). The specimens were loaded through a tension test fixture. This fixture consisted of an expansion cone bearing against four separated, serrated grips located inside the specimen. Radial forces generated by the cone were restrained by a steel band slipped over the casing. Details of the fixture are given elsewhere (16). The free length of all specimens was approx 18 in. (460 mm) between the grips.

The long-term axial creep tests were conducted primarily on two load frames, consisting of 12 test stations/frame. Each test station consisted of a set of compressed load springs, a calibrated load section, the specimen, and a set of fixtures similar to fixtures previously described. The specimen load was checked at intervals no greater than 4 h and adjusted as required.

Axial creep tests were conducted on the basic PVC casing and on each joint configuration. Primary emphasis was placed on the PVC basic casing, PVC bell coupling joint, and ABS threaded casing joint. However, representative tests were also conducted on other configurations. In addition, the effects of a simulated blockage test on the long-term strength of the PVC basic casing were recorded. The simulated blockage fixture for long-term testing consisted of a large C-clamp with a cup placed on one side and one-half of a 10-in. steel pipe welded to the other end. The typical installation procedure was to measure and record all geometric data, assemble the structural joint, allow the joint to cure for a minimum of 24 h, install the specimen in the test fixture, and load the specimen and record the time to failure. In addition, the creep deformation of the specimen was measured and recorded by two dial gages placed on each side of the specimen. The gages were attached to the specimen by two support rings and were placed so an initial gage length of approx 18 in. (460 mm) was obtained.

All specimens were assembled, tested, and conditioned for a minimum of 24 h in an 80° F (27° C) environment. This temperature was selected as representative of a typical casing installation temperature during the summer months. While this temperature is higher than the average ground water temperature for the majority of the country (11), when the entire length of casing was considered, the upper portion of every casing and the lower portion of very deep casings would be at that temperature over a portion of the year. When higher temperatures are expected for a given installation, reduced properties must be used for design.

Joint Test Results

The first study conducted for PVC bell couplings was to evaluate the effect of cure time and environment on the axial strength of the casing joint. Two series of test were completed. For each specimen, the joint was assembled per manufacturer's recommendations. Two coats of primer were applied to the female bell coupling and one coat of primer was applied to the casing. Then, two coats of solvent cement were applied to the casing and one coat of solvent cement was applied to the bell coupling. The two pieces were rapidly assembled

and left undisturbed until tested. The results of the test program are shown in Fig. 2.

Based on the results of this test series, the short-term tensile strength of a cured bell coupling joint is nearly 95% of the basic casing strength. These specimens failed at the neck of the bell coupling or along the solvent-cement bond line. In no case did the specimen fail away from the joint. Failure for all bell coupling joints which failed at a load less than 95% of the basic casing strength occurred along the solvent-cement bond line. Once the allowable shear strength of the solvent cement was exceeded, the strength of the joint significantly decreased. Behavior of the coupling when subjected to long-term axial loads will be considered later.

The result of insufficient cure time in the field could be either the loss of a portion of the casing down the hole or the fracture of the solvent-cement bond and the development of a possible leak path. With proper solvent-cementing procedures and materials, a strong, water-tight joint should result.

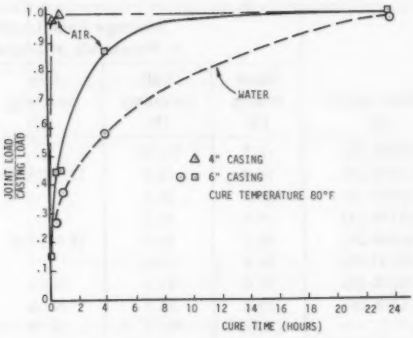

FIG. 2.—Effect of Cure Time on PVC Joint Strength

Based on the results of the first study, all other PVC joint configurations were assembled and cured in an air environment for 24 h before testing. For all solvent-cemented joints, the same procedures previously outlined for the bell coupling cure time tests were followed. The threaded couplings were first solvent-cemented to the casing and permitted to cure overnight. Approximately 2 h before the test, they were threaded together using the solvent cement as a lubricant.

The results of all short-term joint tests are summarized in Table 1. The average short-term axial tensile strength of the bell coupling joint configuration was never less than 87% of the basic strength and the biggest reduction in strength occurred in the 8-in. (200-mm) casing.

The scatter of test data for the slip and threaded couplings was significantly larger than was observed for other tests. However, the failures tended to fall into two categories. The failure mode of the lower-strength specimens was a shear failure of the solvent-cement bond line. The higher-strength failure loads resulted from coupling failures. For some of the fittings, the absence of a tight,

interference fit between the casing and coupling could explain some of the strength variation.

In general, the slip couplings were weaker than the basic casing for the wall thicknesses tested. The strength of the threaded couplings was also generally below the basic casing strength. In fact, no 6-in. (150-mm) threaded coupling developed the full casing capability. These coupling configurations do develop sufficient strength for use in many water well casing applications. However, additional handling constraints may be required for some installations.

Typical results from the simulated blockage tests on 6-in. (150-mm) casing are presented in the following. The peak load at 13% blockage using casing from Manufacturer C-6 (SCH-40) for the basic pipe was 2.2 kip, the bell coupling was 1.94 kip, the slip coupling was 6.14 kip, and the threaded coupling was 6.4/9.4 kip. For each joint configuration, the peak load required to push the

TABLE 1.—PVC Axial Strength After 24-h Cure Time

Nominal size, in inches (1)	Manufacturer (2)	Average Axial Load, in thousands of pounds			
		Basic casing (3)	Bell coupling (4)	Slip coupling (5)	Threaded coupling (6)
4	A(SDR-26)	16.3	N/A		
	B(SDR-26)	16.4	16.4	12.7–16.5[a]	12.8–22.0[b]
	C(SCH-40)	21.5	21.3		
6	A(SDR-21)	39.6	39.8		
	B(SDR-26)	36.0	36.0	18.6–32.0[c]	15.9–20.2[d]
	C(SCH-40)	38.9	38.0		
8	A(SDR-26)	57.0	49.5	N/A	N/A
	B(SDR-26)	59.8	54.5	N/A	N/A
	C	N/A	N/A	N/A	N/A

[a]Average = 14.5 kip.
[b]Average = 15.0 kip.
[c]Average = 21.6 kip.
[d]Average = 18.4 kip.
Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm; N/A = not available.

joint past an obstruction equal to 13% of the nominal casing diameter was not significant. However, for each configuration, the water-tight integrity of the joint was significantly impaired. That is, at the conclusion of each test, a clear, open leak path was present so possible contamination of the ground water could occur.

Therefore, clean straight bore holes should be provided before the installation of thermoplastic water well casing begins. It would appear that the casings could be easily pushed past a reasonably sized obstruction. However, the worst effect of the obstruction would be on the integrity and water tightness of the joint and could have significant environmental consequences.

The only ABS joint configuration tested was the threaded casing. ABS casings can be belled but none were tested. It would be expected that the results for

ABS bell couplings would be quite similar to the PVC bell coupling results. However, a different primer/solvent-cement system especially formulated for ABS would be required.

The threaded casing was supplied in an SDR-17 wall thickness and 4.5-in. (110-mm) and 6-in. (150-mm) nominal diam. Because of the thick walls, the threads were cut directly into the end of the casing. Thus, a smooth wall was provided inside and outside of the joint. The thin metal ring provided on the outside of the casing end with the female threads did not appear to improve the strength or the structural performance of the joint.

In the specimens provided, adjacent sections were easily threaded together. The joint installation procedure was to hand tighten the joint until snug, mark both sections, and further tighten the joint with heavy wrenches. For each casing size, one set of specimens was tightened until they failed. With three men, it was impossible to break the 4.5-in. (110-mm) casing joint with the wrenches. However, the 6-in. (150-mm) joint did eventually break after significant abuse.

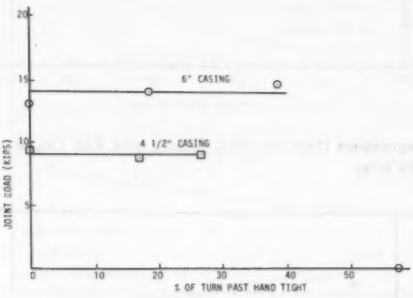

FIG. 3.—Effect of Torque on Threaded ABS Casing Joint (1 kip = 4.45 kN)

In practice, reasonable installation procedures would not permit the abuse given to these specimens.

After the specimen was tightened to the desired position, a short-term tension test was conducted and the peak load was recorded. The results of this study are presented in Fig. 3. Analysis of the data indicates that the effect of torque on the threaded configuration does not significantly affect the axial load capability of the joint configuration. For each specimen, the failure occurred in the first thread cut at the base of the female end. This mode is typical of all threaded connections and is not significant. Since the joint is machined from the parent casing wall, the axial strength of the joint is not as high as the basic casing strength. However, it does possess sufficient strength so that the allowable dry string length is very high. This is more than adequate for most field applications.

A properly formulated design criteria for casing joints must include the inherent stress degradation properties of thermoplastic materials. Since each joint configuration was made by changing the geometry of the basic casing or shearing the load into a coupling, significant secondary stresses would be introduced

into the joint. Because the long-term strength of thermoplastic materials is very stress dependent, it would be wrong to develop a design criteria based on short-term tests alone.

To assess the adequacy of the load frame, test procedures, and fixture attachments, the first long-term tests were conducted on specimens made from

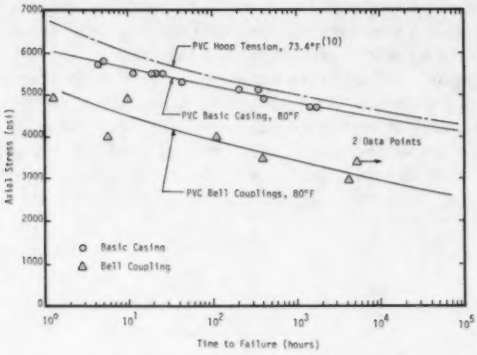

FIG. 4.—Stress Regression Data for PVC Casing and Bell Coupling Joints at 80° F (27° C) (1 psi = 6.89 kPa)

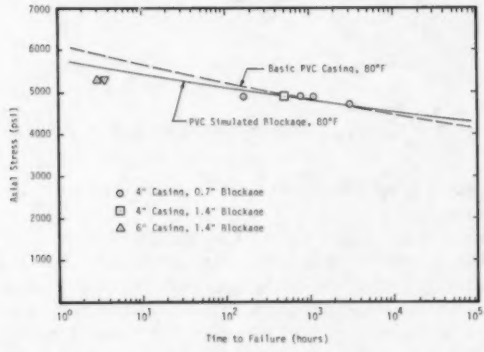

FIG. 5.—Stress Regression Data for PVC Casing with Simulated Blockage (1 psi = 6.89 kPa)

basic PVC casing. As can be seen in Fig. 4, the stress regression data compared quite favorably with existing data developed from room-temperature pressure tests (10). Since the casing tests were conducted above room temperature, it would be expected that the time to failure at a given stress level would be lower than the existing room-temperature failure time. It also appears that the effects of residual stresses in the circumferential or axial direction and casing

manufacturer on long-term strength are small.

The axial creep tests were repeated except that the specimens were also subjected to a simulated hole blockage. The amount of blockage ranged from 0.7 in. (18 mm)–1.4 in. (36 mm) from the casing outside diameter. The stress regression data are presented in Fig. 5. When compared to the basic casing data, it is seen that the simulated blockage significantly reduced the time to failure at high stress levels but did not impact the time to failure at lower stress levels. While the simulated blockage did not significantly affect the strength of the casing, it did accelerate the necking induced in the casing. This necking could cause serious operational problems over the life of the casing. Therefore, any type of hole blockage should be avoided in the field.

The PVC bell couplings were assembled as previously described and installed in the load fixture. The stress level was determined by dividing the axial load by the cross-sectional area of the basic casing away from the joint. Because

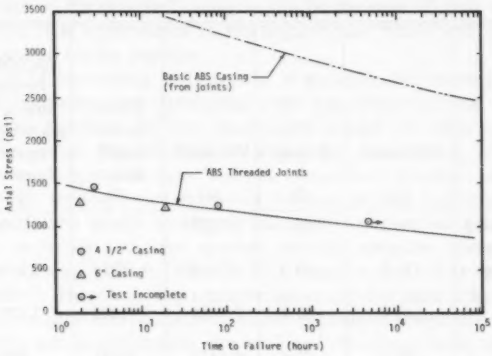

FIG. 6.—Stress Regression Data for ABS Threaded Casing Joints and ABS Basic Casing at 80° F (27° C) (1 psi = 6.89 kPa)

of the geometry of a bell coupling, additional bending stresses are induced locally in the coupling. The data were also presented in Fig. 4. The failure mode was a fracture on the female end of the joint in the transition area between the basic casing and the bell. The local bending stresses in this area impacted the long-term strength of the casing system. Therefore, the long-term strength of the bell coupling becomes a critical design parameter of a thermoplastic casing system.

ABS threaded casing joints were also tested for long-term strength properties. The stress regression data for this joint configuration are presented in Fig. 6. Again, the stress level was determined by dividing the total axial load by the cross-sectional area of the basic casing. Because the threads were cut directly into the casing wall, the actual stresses at the joint are significantly higher than those calculated.

The stress regression data for the basic ABS casing material were determined by multiplying the ABS joint data by the ratio of basic casing area and the

effective joint area. The effective joint area was determined by dividing the short-term joint strength by the short-term tensile strength of the material at yield [5.04 ksi (34.7 MN/m^2)] (7). The failure mode of each joint configuration was at the first thread of the male or female end. Very little plastic creep was observed during or prior to failure.

For each set of experimental data, a regression analysis was conducted. A predicted regression equation of the form $\log t = A + B \log \sigma$, in which t = time to failure, in hours; σ = axial stress, in pounds per square inch; and A and B = constants, was utilized. Constants A and B were determined by the least-squares technique. In addition, the correlation coefficient, r, and variance, s^2, were also calculated.

The approach used to determine the allowable pressure rating of the thermoplastic pressure rated pipe outlined in ASTM Standard D2837 (15) was used

TABLE 2.—Regression Analyses Summary

Data type (1)	Intercept, A (2)	Slope, B (3)	Variance, s^2 (4)	Correlation coefficient, r (5)	100,000-h failure stress, in pounds per square inch (6)	Design stress, in pounds per square inch (7)
Basic PVC casing	110.2	−29.1	0.02295	−0.988	4,100	2,050
PVC (A11) simulated blockage	155.7	−41.5	0.2588	−0.954	4,280	2,140
PVC bell couplings	63.4	−17.1	0.4669	−0.902	2,625	1,310
ABS threaded joints	71.0	−22.4	0.5412	−0.917	890	445
ABS basic casing (from joint)	90.0	−25.1	0.0455	−0.993	2,470	1,235

Note: 1 psi = 6.89 kN/m^2.

to determine the allowable design stresses for casing joints. Basically, the approach is to estimate the allowable stress from the least-squares prediction equation after 100,000 h (11.4 yr) of load. Then, a factor of safety of 2 is applied to the predicted failure stress to determine the allowable design stress. A summary of the analysis is presented in Table 2.

The allowable dry hole length of casing can be determined by dividing the allowable design stress by the unit weight of the casing material. For PVC (s.g. = 1.4) casing with bell couplings and threaded ABS (s.g. = 1.04) casings, the allowable dry hole length is approx 2,160 ft (658 m) and 980 ft (300 m), respectively. These large lengths show that thermoplastic casing materials and systems have more than enough long-term strength for applications in water wells. These lengths are conservative in many applications because, in practice, the casing may eventually reach the bottom of the bore hole due to the axial creep and the applied stresses will tend to relieve themselves.

Summary and Conclusions

The results of the short-term joint tests indicated that several different joint configurations have properties suitable for water well casing applications. No one system was found to be the best configuration for all conditions. The solvent-cemented bell coupling is a very simple configuration but requires the use of solvent cements. While appropriate for most situations, some environments would require longer cure times than those experienced during the test program. In addition, the strength of the resultant joint is dependent upon the procedures and materials used to make the joint. The use of the slip couplings would be appropriate for installation or repair of casing during installation. The threaded couplings can be solvent cemented to the casing before being transported to the job site. They are then assembled by threading the adjacent sections. However, the strength of the slip and threaded couplings was significantly lower than the basic casing strength.

The advantage of the threaded casing would be the ease of field installation. However, where field applications do not require such heavy wall thicknesses, a lighter wall would not be possible.

The behavior of the casing joint system when subjected to long-term axial loads must be evaluated when developing a rationale design criteria. The impact of the simulated blockage did not significantly impact the time to failure at the lower stress levels. The blockage did encourage the tendency of the casing to neck. This necking could cause serious operational concerns in the field. Suggested design stresses were proposed for two casing materials and joint configurations. Based on the results of this study, the commercially available thermoplastic materials and joint systems provide adequate strength for dry string lengths of over 1,000 ft (300 m). This length is more than adequate for most applications. However, the engineer must not overlook other important design conditions such as hydrostatic collapse pressures, temperatures, installation procedures, etc. In conclusion, the results of this study verify that thermoplastic well casing systems have inherent properties which make them suitable for well casing applications.

Acknowledgments

This study was supported in part by the United States Environmental Protection Agency Project Number R805015 administered through the Water Resources Research Institute of Auburn University. The materials supplied by the casing manufacturers are gratefully acknowledged.

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California Irrigation Return Flow Case Studies[a]

By Kenneth K. Tanji[1]

Introduction

This paper presents information and data on the supply and surface drainage waters from the 66,420-ha (164,000-acre) Glenn-Colusa Irrigation District and the 17,820-ha (44,000-acre) Panoche Drainage District for 1975, a typical irrigation season (6). Differences are noted in the quantity and quality of irrigation return flow from these two districts caused by site specific conditions and factors. The impacts of these irrigation return flows on the quantity and quality of the Sacramento and San Joaquin Rivers are also appraised. The case study information and data are then considered in the light of Section 208 (Public Law 92-500) Water Quality Management Planning now being considered for nonpoint sources of pollutants in the Central Valley of California.

Case Studies

General Description of Districts.—Fig. 1 shows the location of the Glenn-Colusa Irrigation District (Glenn-Colusa) and the Panoche Drainage District (Panoche). Glenn-Colusa is the oldest (1887) and largest [66,420-ha (164,000-acre)] irrigation district in the Sacramento Valley. The soils in Glenn-Colusa along the Sacramento River and creeks from the Coast Range Mountains are coarser-textured while soils in the basin and basin rim are finer-textured, some of them sodium-affected. The growing season is about 224 frost-free days. Precipitation occurs mainly in the winter with mean annual precipitation of about 460 mm (18 in.).

Table 1 shows that paddy rice is the major crop which is flooded continuously throughout the growing season. Pastures, hay crops, and orchards are usually irrigated by the basin or border method. Row crops such as corn, tomatoes, and sugar beets are furrow irrigated. Less than 0.5% of the total 56,630 ha

[a]Presented at the July 17–20, 1979, ASCE Specialty Conference on Irrigation and Drainage in the Nineteen-Eighties, held at Albuquerque, N.M.

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on October 15, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0209/$01.00.

(139,800 acre) of irrigated land is sprinkler irrigated. The irrigation season starts in April and ends in October. In addition to crop irrigation, Glenn-Colusa delivers water to private duck clubs and the Federal Sacramento National Wildlife Refuge which is devoted mainly to migratory birds on the Pacific Flyway.

The present organization of the 17,820-ha (44,000-acre) Panoche Drainage District was formed in 1947, but the lands were previously irrigated by the

FIG. 1.—Location Map for Case Study Areas

Miller-Lux Canal built in 1871. Because the soils in Panoche are derived from marine sedimentary rocks they vary from coarser-textured, relatively salt-free surface soils in the upper alluvial fans to finer-textured, salt-affected soils in the lower alluvial fans and the basin. These salt-affected soils contain moderate to high concentrations of soluble salts, gypsum, boron, and nitrates. Tile drainage

is increasingly being practiced to control the rising water tables and to leach soluble salts out of the root zone. Panoche has about 323 frost-free days and receives about 200 mm (8 in.) of mean annual precipitation.

Table 1 indicates that the major crops are cotton, tomatoes, sugar beets, and cantaloupes and other melons which are grown in the summer and are furrow irrigated. Barley, wheat, and lettuce are grown in the winter. About 15% of the 16,160 ha (39,900 acre) of irrigated land is sprinkler irrigated for

TABLE 1.—1975 Cropping in Case Study Areas

Glenn-Colusa Irrigation District		Panoche Drainage District	
Crops (1)	Area, in hectares (acres) (2)	Crops (3)	Area, in hectares (acres) (4)
Rice	37,790 (93,320)	Cotton	4,460 (11,000)
1- and 2-irrigation crops	4,210 (10,400)	Tomatoes and sugar beets	3,470 (8,580)
Pasture and alfalfa	3,030 (7,490)	Small grains and safflower	2,920 (7,220)
General field crops	2,550 (6,290)	Melons	1,660 (4,100)
Tomatoes, sugar beets, and corn	1,850 (4,580)	Pasture	500 (1,240)
Decidious nut and fruit trees	730 (1,800)	Lettuce and garlic	490 (1,210)
Wildlife refuge duck ponds	1,260 (3,100)	Alfalfa and vegetable seeds	410 (1,010)
Wildlife refuge cultivated grain	1,240 (3,050)	Alfalfa hay	190 (470)
Private duck ponds	970 (2,390)	Almonds and prunes	100 (240)
		Others[a]	1,960 (4,850)
Total	53,630 (132,420)		16,160 (39,920)

[a]Crop records available only from Panoche Water District. Three other small districts have about 4,850 acre of rice, pasture, and small grains.

seed germination in furrow beds. Some border irrigation is practiced for the close-growing crops. The irrigation season usually starts in March and ends in February of the following year. The winter crops may receive some preirrigation or irrigation, or both, depending on the fall and winter rainfall.

Water Supply and Drainage.—Fig. 2 shows the Glenn-Colusa Irrigation District. In 1975 this district obtained 877×10^6 m^3 (711,000 acre-ft) of supply water by diverting Sacramento River water with low lift pumps near Hamilton City as well as 153×10^6 m^3 (124,000 acre-ft) of water diverted by gravity from

Stony Creek. Glenn-Colusa has riparian water rights to the Sacramento River and purchases additional water from the United States Bureau of Reclamation (now designated as the United States Water and Power Services) Central Valley Project amounting to about 10% of their total supply.

The supply water is distributed by the main canal that has a 85 m^3/s (3,000 cfs) capacity and is 105 km (65 mile) in length. There are some 644 km (400 mile) of lateral distribution canals. All of these canals are unlined but with permanent concrete structures. Flow is by gravity. The distribution system also involves capture of drain waters by either gravity or low lift pumps at 20 locations scattered throughout the district. About half of the surface drain waters are reused [274×10^6 m^3 (222,000 acre-ft)] within the district and the remainder is discharged into the upper reaches of the Colusa Basin Drain via 11 lateral drains. The district inflow monitoring station is at D14 and the outflow station, D13 (Fig. 2).

The Colusa Basin Drain conveys flood runoffs and irrigation return flows from a 405,000 ha (1,000,000 acre) drainage area of which some 35% are devoted to irrigated agriculture. Extensive reuse is made of drain waters in the Colusa Basin Drain by irrigated agriculture and Federal wildlife refuge units. A portion

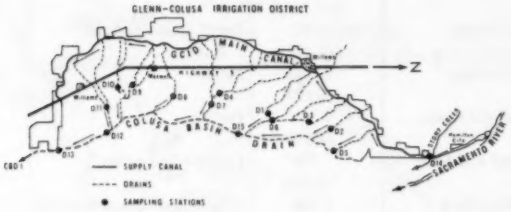

FIG. 2.—Glenn-Colusa Irrigation District

of the flow in the 113-km (70-mile) long Colusa Basin Drain is discharged into the Sacramento River above Knights Landing.

Fig. 3 shows the supply and drainage network for the Panoche Drainage District. This district had previous water rights to the San Joaquin River, which was later impounded upstream by the Friant Dam as part of the Central Valley Project, for water users in the east side of the San Joaquin Valley. In exchange for the San Joaquin River water, Panoche now receives water imported from the Sacramento Valley through the older, unlined Delta-Mendota Canal and the newer, concrete-lined California Aqueduct. In 1975, about 107×10^6 m^3 (87,000 acre-ft) of water from the former canal were lifted at two stations by a series of pumps up to as much as 46 m (150 ft), while 51.6×10^6 m^3 (41,900 acre-ft) of water from the latter canal were diverted by gravity from three stations (Fig. 3). The distribution system consists of 82 km (51 mile) of canals, 29 km (17 mile) of which are small, concrete-lined laterals.

Panoche and a contiguous district, Central California Irrigation District, captured 1.80×10^6 m^3 (1,500 acre-ft) of drain waters produced within Panoche, and the remaining 48.9×10^6 m^3 (39,600 acre-ft) was discharged into the Grasslands Water District through Panoche Drain. The Grasslands Water District uses this

return flow for irrigating pastures as well as maintaining a wetland habitat for migratory and resident birds. The remainder of the unused drain waters from Panoche Drain are comingled with other return flows from adjacent areas in Mud Slough and are further reused by downstream water users before ultimate discharge into the middle reaches of the San Joaquin River via Salt Slough, upstream from the confluence of Merced River.

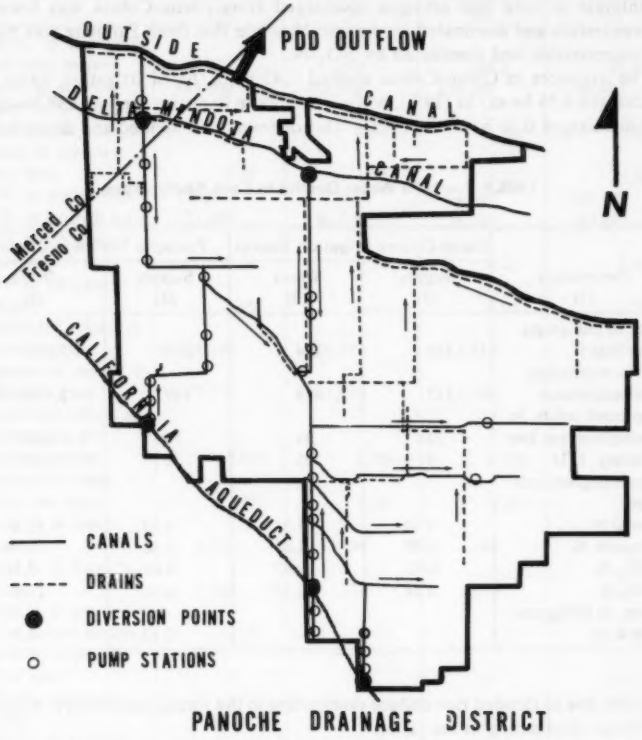

FIG. 3.—Panoche Drainage District

Surface Irrigation Return Flows.—Table 2 summarizes the flow weighted average concentrations of selected quality parameters in supply and drain waters from Glenn-Colusa and Panoche, while Table 3 gives the mass (concentration × volume) inflow and outflow data for water quality parameters.

The supply water in Glenn-Colusa (Table 2) is of good quality, and the drain waters have not been appreciably degraded with respect to salinity, suspended matter and nitrogen. The irrigation return flow was comprised mainly of surface runoff from rice fields. In contrast, the supply water in Panoche was similar

in quality to that of drain waters from Glenn-Colusa, and the drain waters from Panoche were considerably degraded as a result of the collection of tile drainage effluents from salt-affected lands. It is not certain how long these high concentrations of total dissolved solids (TDS), NO_3-N, and boron will persist in tile effluents from newly installed drains. The moderate concentration of suspended solids comes from the highly erodible Panoche soil series, which is furrow irrigated, as well as from erosion of banks of drain channels. It is of interest to note that nitrogen discharged from Glenn-Colusa was lower in concentration and dominated by organic N, while that from Panoche was higher in concentration and dominated by NO_3-N.

The irrigators in Glenn-Colusa applied 1.92 ha-m/ha of irrigation water and discharged 0.56 ha-m/ha (29%), while irrigators in Panoche applied 0.98 ha-m/ha and discharged 0.30 ha-m/ha (31%). This difference in applied and drain waters

TABLE 2.—1975 Water Quality in Case Study Areas

Parameters (1)	Glenn-Colusa Irrigation District		Panoche Drainage District	
	Supply (2)	Drain (3)	Supply (4)	Drain (5)
TDS, in milligrams per liter	116	244	219	2,050
EC, in micromhos per centimeter	117	418	389	2,780
Suspended solids, in milligrams per liter	24	36	75	348
Turbidity, JTU	9	25	31	125
N, in milligrams per liter				
Total N	1.04	1.31	1.13	13.4
Organic N	0.87	1.05	0.46	0.70
NH_4-N	0.03	0.07	0.05	0.10
NO_3-N	0.14	0.19	0.62	12.6
Boron, in milligrams per liter	—	—	0.12	4.3

is mainly due to flooded rice culture dominating in the former and furrow irrigated row crops dominating in the latter.

On a unit mass basis, the TDS in the applied water was about 0.22 kg/m^2 for both districts (Table 3) while that discharged was 0.13 and 0.61 kg/m^2 from Glenn-Colusa and Panoche, respectively. In the former, there was a net loss of TDS, some of which was presumably lost through deep percolation since accumulation of salts is not observed; and in the latter, there was a net gain of TDS largely caused by the chemical weathering of gypsum present in the soils (5).

Although the concentration of suspended solids in Glenn-Colusa was slightly higher in the drain water as compared to the supply water, the unit mass emission of suspended solids was only about half because the volume of drain water was smaller. On the other hand, the concentration of suspended solids in the

drain waters of Panoche was 4.6 times greater than the supply water, but the unit mass emission was only 1.4 times greater. This difference may be attributed

TABLE 3.—1975 Mass Flows of Water and Quality Parameters

Parameters (1)	Glenn-Colusa Irrigation District		Panoche Drainage District	
	Supply (2)	Drain (3)	Supply (4)	Drain (5)
Water, in thousands of cubic meters	1,030	300	158.8	48.8
Water, in acre-feet	835,400	243,500	128,800	39,600
Water, in hectare-meters per hectare	1.92	0.50	0.98	0.30
Water, in acre-feet per acre	6.31	1.84	3.21	0.99
TDS, in metric tons	119,570	73,300	34,810	100,370
TDS, in English tons	131,800	80,800	38,370	110,640
TDS, in kilograms per square meter	0.22	0.13	0.21	0.61
TDS, in tons per acre	1.00	0.61	0.96	2.76
Suspended solids, in metric tons	24,740	10,810	11,910	17,020
Suspended solids, in English tons	27,270	11,920	13,130	18,760
Suspended solids, in kilograms per square meters	0.046	0.020	0.073	0.10
Suspended solids, in tons per acre	0.21	0.09	0.33	0.47
Total N, in metric tons	1,070	394	180	656
Total N, in English tons	1,182	434	198	727
Total N, in grams per square meters	2.02	0.67	1.12	4.03
Total N, in pounds per acre	18	6	10	36
NO_3-N, in metric tons	144	57	99	6.16
NO_3-N, in English tons	159	63	109	679
NO_3-N, in grams per square meters	0.27	0.11	0.60	3.79
NO_3-M, in pounds per acre	2.4	1.0	5.4	33.8

to the fact that flooded rice fields in Glenn-Colusa more or less act as settling basins for suspended solids, while the soils in Panoche are highly erodible.

In Glenn-Colusa, the concentration of NO_3-N discharged was 1.4 times greater than the supply water but the unit mass emission was only 0.3 times as much.

Conversely, the NO_3-N in drain waters from Panoche was 20.3 times greater in concentration and 6.3 times greater on a mass emission basis. The main source of NO_3-N from Panoche is of geologic origins while that from Glenn-Colusa is decomposition of crop residue and algae from flooded rice fields.

The preceding examples point out that both concentration and unit mass emission need to be considered when evaluating surface irrigation return flows, and they vary between these two particular districts as a result of site specific conditions and factors (6).

Quantity of return flows is influenced by: (1) Availability and cost of supply water; (2) irrigation application methods and efficiencies; (3) extent of reuse at the on-farm, district, and basin levels; (4) special cultural practices such as seed germination in furrow beds and lowering of flood waters in rice fields during gusty winds; and (5) constraints on reuse due to the presence of excessive concentrations of boron, sodium, and chloride. Quality of return flows is influenced by: (1) Quality of the supply water and reuse of drain waters; (2) presence of salts, boron and nitrogen native to the soils; (3) leaching fraction and the mineral precipitation-mineral dissolution phenomena; (4) use of agricultural chemicals and wastes such as gypsum and animal manures; (5) erodibility of surface soils and banks of open channels; (6) relative proportion of surface and subsurface return flows; and (7) discharges into irrigation drains by other sectors of society (7).

Receiving Water Quality.—As mentioned earlier herein, the surface irrigation return flows from Glenn-Colusa and Panoche are not discharged directly into surface receiving water bodies. In view of these complex drainage systems, Table 4 compares the quality and flow of the receiving waters along with the comingled drain outfalls, as well as the case study district outflows.

Because there are comparatively large flows in the Sacramento River, the contributions of flow and quality inputs by Glenn-Colusa and other dischargers into Colusa Basin Drain are insignificant. However, it should be pointed out that a turbid plume is usually visible at the outfall of Colusa Basin Drain near Knights Landing. In contrast, the middle reaches of the San Joaquin River during the irrigation season are largely filled with irrigation return flows from the east and west sides of the San Joaquin Valley. The quality of the east side return flows, however, is generally better than those from the west side, since the soils in the former area are nonsaline, and are derived from granitic alluvium from the Sierra Nevada Mountains.

Table 4 shows that Salt Slough contributed 61% of the flow in the San Joaquin River monitored at Fremont Ford, 79% of the TDS, 54% of the suspended solids, and 147% of the NO_3-N. The NO_3-N discharged is assimilated by phytoplankton and other aquatic plants and/or denitrified to such an extent that considerable reduction in concentration takes place. The TDS discharged from Panoche accumulates in the basin soils or precipitates as insoluble salts in the Grassland Water District and other units that reuse the irrigation return flows. Similarly, the suspended solids discharged from Panoche are deposited to some extent in channels and wetland habitats as flow velocity decreases.

Beneficial Uses of Receiving Waters.—The Sacramento and San Joaquin River systems each have multiple and competing beneficial uses (9). Waters of the Sacramento River are heavily exported into the San Joaquin Valley and the Los Angeles Basin through the Central Valley Project and the California State

Water Project for agriculture and municipal-industrial water uses. Water flowing into and through the Sacramento-San Joaquin Delta repels seawater intrusion and maintains a freshwater habitat for anadromous fish and other fish species.

The specific beneficial uses of water in the Sacramento River from the Colusa Basin Drain outfall to the confluence of the American River at the city of Sacramento include: (1) Municipal and domestic water supply; (2) irrigated agriculture; (3) recreation such as water skiing, boating and sport fishing; (4) freshwater habitat for warm water fish such as sturgeon, catfish, shad, and

TABLE 4.—1975 Return Flow Inputs on Receiving Streams

	Sacramento River			San Joaquin River		
Parameters (1)	Glenn-Colusa irrigation district outflow (2)	Colusa Basin drain outfall (3)	Sacramento River below Knights Landing (4)	Panoche drainage district outflow (5)	Salt slough outfall (6)	San Joaquin River at Fremont Ford (7)
Water, $10^6 m^3$	300	267	5,218	48.8	56.7	93.7
Water, in acre-feet	243,500	216,560	4,232,000	39,600	46,020	76,000
Water, percentage contribution	6	5	—	52	61	—
TDS, in milligrams per liter	244	427	125	2,050	821	627
TDS, in metric tons	73,300	114,090	652,680	100,370	46,610	58,800
TDS, in English tons	80,800	125,760	719,440	110,640	51,380	64,810
TDS, percentage contribution	11	17	—	171	79	—
Suspended solids, in milligrams per liter	36	111	66	348	91	103
Suspended solids, in metric tons	10,810	29,660	344,610	17,020	5,170	9,660
Suspended solids, in English tons	11,920	32,690	379,860	18,760	5,700	10,650
Suspended solids, percentage contribution	3	9	—	176	54	—
NO_3-N, in milligrams per liter	0.19	0.14	0.09	12.6	1.75	0.73
NO_3-N, in metric tons	57	37	470	616	100	68
NO_3-N, in English tons	63	41	518	679	110	75
NO_3-M, percentage contribution	12	8	—	906	147	—

striped bass; (5) cold water fishery habitat and migration of salmon and steelhead; (6) wildlife habitat; and (7) navigation. Similar beneficial uses are made of the San Joaquin River from the mouth of the Merced River to Vernalis. This reach of the San Joaquin River also has other beneficial uses such as process water for industry, stock watering and freshwater habitat for migration and spawning of cold water fish.

Other than the kinds of water quality data reported in Table 4, definitive investigations to ascertain impacts of irrigation return flows on the beneficial

water uses of the Sacramento River are rather limited. Hayes, et al. (2) reported that the density of periphyton (attached algae) in the Sacramento River below the drain outfall was higher than above the outfall due to nutrient inputs from the drain. Tanji, et al. (8) have determined that the suspended matter in the Colusa Basin Drain is composed of 56% mineral (sediments), 8% algae, and 36% nonliving biomass.

In contrast, the irrigation return flows, particularly tile drainage effluents from the west side of the San Joaquin Valley, have been extensively monitored (4) and studied (1) with respect to salinity and nitrogen. Additional studies on the impact of irrigation return flows on aquatic organisms are underway.

To protect the competing and multiple beneficial uses of the Sacramento and San Joaquin Rivers, the state of California has established stream water quality objectives (standards) at Freeport and at Vernalis, respectively (9).

Section 208 Planning

The State Water Resources Control Board in 1975 prepared a water quality control plan for each of the 16 river basins in California. As a continuation of the planning process required by Section 208 of PL 92-500, water quality management planning is being performed in seven designated areas (Lake Tahoe, Sacramento, San Francisco, Monterey, Ventura, South Coast, and San Diego) and five nondesignated areas (North Coast, Central Coast, Central Valley, Lahontan, and Colorado River Basin). The 1975 river basin plans and the ongoing designated area 208 Planning are concentrated on control of point sources of pollutants. The program for nondesignated areas is focused on developing recommendations for management of nonpoint sources of pollution. The approach taken involves problem identification and development and implementation of best management practices (BMP's).

The State Water Resources Control Board (10) identified four priority planning issues for agriculture: (1) Soil erodibility and sediment production; (2) salinity drainage in the San Joaquin Valley; (3) impact of agricultural drains on aquatic and wildlife resources in the Colorado River Basin; and (4) pesticide residues in return flows. In the Central Valley Nondesignated Area 208 Planning that includes the case study districts, the Regional Water Quality Control Board has delegated management planning on sediment problems to the Soil Conservation Service. Eight pilot study areas in the Central Valley have been studied. A preliminary draft report by the Soil Conservation Service (3) gives the recommended plan of BMP for agricultural sediments. This plan advocates an integrated "Resource Management System" for specific land uses (e.g., grazing land, cropland, wildlife land) with a particular set of resource management practices (e.g., planned grazing system, range seeding, grassed waterway) for treatment of site specific problems (e.g., sheet and rill erosion, gully erosion, streambank erosion). For the problem of agricultural drainage and salt management in the San Joaquin Valley, an interagency study group has prepared a preliminary edition of a final report (1). This study group recommends construction of a concrete-lined canal along the west side of the valley to convey salt laden drainage waters to the Western Delta for ultimate disposal into the Pacific Ocean. It is recommended that about 22,200 ha (56,000 acre) of marshland be maintained and 6,500 ha (16,000 acre) of storage reservoir be built to provide operational

flexibility. Underdrainage is required to control water logging and salt accumulation in the crop root zone of some 162,000 ha (400,000 acre) of irrigated land. This plan is receiving mixed public reaction. The solution to and control of sediments, salts, and nutrient problems in the Central Valley are now under review.

Conclusions

The evaluation of irrigation return flows from two typical irrigation districts in the Central Valley of California indicates that the quantity and quality of surface return flows are highly variable as a result of site specific factors and conditions. The impacts of these return flows on surface receiving water bodies may also vary. It is suggested that due considerations be given to such site specific factors and conditions when developing best management practices and water quality control plans.

Acknowledgment

This investigation was supported by EPA Grant No. R803603. The writer acknowledges the assistance of Robert Clark, Glenn-Colusa Irrigation District, and Ray Ram, Panoche Drainage District.

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Appendix II.—Notation

The following symbols are used in this paper:

EC = electrical conductivity;
JTU = Jackson Turbidity Units;
NH_4-N = ammonium-nitrogen;
NO_3-N = nitrate-nitrogen;
TDS = total dissolved solids; and
TOTAL N = total nitrogen.

GRAND VALLEY IRRIGATION RETURN FLOW CASE STUDY

By John W. Keys, III,[1] M. ASCE

INTRODUCTION

Purpose.—The ASCE Task Committee on the Evaluation of Water Quality Regulatory Programs Affecting Agricultural Return Flows in Arid Areas of the United States was established by the Irrigation and Drainage Division in 1976. The objectives of this Task Committee were as follows:

1. To evaluate ongoing Federal, state, and local programs pertaining to the water quality aspects of agricultural return flows in the arid, western areas of the United States, specifically analyzing the effects of these programs on local irrigated areas.
2. Based on the preceding evaluation, to develop equitable, feasible, and appropriate recommendations for management plans relating to point and nonpoint agricultural return flows.

The purpose of this paper is to present a case study of irrigation in the Grand Valley of Colorado to show the magnitude of irrigation in the area, its influence and impact on the area and its streams, some of the laws and regulations it must operate under, and the actions being implemented to control the quality of irrigation return flows.

Description of Area.—Grand Valley is located in west-central Colorado in the Colorado River Basin. It is about 12 mile (19 km) wide and 35 mile (56 km) long and extends from DeBeque Canyon on the east into Utah on the West. The Colorado River flows through the valley and is joined by the Gunnison River near the city of Grand Junction.

Grand Valley is an arid region with an average annual rainfall of only 8.3 in. (210 mm). Temperatures in the valley have ranged from −23° F–105° F (−31° C–41° C) with an annual mean of 52° F (11° C), and July and January means of 78° F and 26° F (26° C and −3° C), respectively. The average growing

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on October 15, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0221/$01.00.

season is 190 days long. Elevation of the valley is approx 4,700 ft (1,400 m) above mean sea level.

The population of Grand Valley is estimated at 63,500, with most of the people located in the vicinity of Grand Junction. Other communities in the valley are Palisade, Clifton, Fruita, Loma, and Mack. These towns, along with the rivers and irrigated land of the valley, are shown on the map of Fig. 1.

Agriculture (primarily cash and forage crops and livestock production), is the principal industry in Grand Valley, with approx 71,500 acre (28,900 ha) of irrigated cropland. Corn, small grains, and alfalfa are the main crops, although fruit production is also significant in the eastern end of the valley. Many farms also serve as a base for livestock operations, where winter feed is grown and cattle and sheep are wintered. Most of these animals are moved to public and private grazing lands surrounding the valley during the warmer months.

Geologically, Grand Valley was eroded by the Colorado River from Mancos Shale, a thick sequence of beds of marine origin which underlies much of the

FIG. 1.—General Map, Grand Valley, Colorado

valley. The shale contains a high percentage of salts, and gypsum is commonly found in open joints and fractures. The formation is generally impervious, but a weathered zone near the surface transmits water along joints, fractures, and bedding planes.

Soils of the Grand Valley are generally derived from the Mancos Shale formation. Strongly saline Cretaceous formations dominate the parent material of the area, with approx 78% of the area having soils directly related to the Mancos Shale. The shallow Chipeta and Persayo soils and the moderately deep phases of the Fruita and Mesa soils resting on Mancos Shale amount to 31% of the area. The deep Billings, Mack, and Ravola soils developed in alluvium from the Mancos Shale comprise about 47% of the area. The deep Fruita, Genola, Green River, Hinmon, Mesa, Naples, and Thoroughfare soils which are formed in mixed material of other than Mancos Shale amount to about 22% of the area.

Irrigation in Grand Valley

Irrigation in Grand Valley has been underway since about 1882 when the first settlers arrived in the valley and began work on ditches to water lowlands adjacent to the north side of the Colorado River. By 1886, the Grand Valley Canal was completed and the canal system expanded to serve approx 45,000 acre (18,200 ha) of land. From 1886–1902, several attempts were made by private interests to construct a canal to higher lands in the valley, but technical and financial problems prevented the development. With passage of the Reclamation Act of 1902, studies began on a project to serve those lands. Construction of the Grand Valley Project facilities began in 1912, with completion of irrigation facilities in about 1926. Since it first delivered water in 1917, the Grand Valley Project has furnished a full supply of irrigation water to about 42,000 acre (17,000 ha) of land in the valley. Crops produced on all irrigated lands of Grand Valley are shown in Table 1.

TABLE 1.—Crops Produced on Irrigated Lands of Grand Valley

Crop (1)	Percentage of irrigated area (2)
Alfalfa	25
Corn	25
Other small grains	10
Irrigated pasture	10
Fruit	8
Hay	3
Seed production	1
Idle land	18

Water Supply.—The primary sources of water in Grand Valley are the Colorado and Gunnison Rivers and Plateau Creek. Annual inflow to the valley is about 4,500,000 acre-ft ($5,500 \times 10^6$ m^3) with an outflow of about 4,300,000 acre-ft ($5,300 \times 10^6$ m^3), as shown in Table 2.

Water for irrigation in Grand Valley is diverted from the Colorado and Gunnison Rivers. The total annual diversion of water in the Grand Valley is about 1,590,000 acre-ft ($1,960 \times 10^6$ m^3), but this includes about 960,000 acre-ft ($1,200 \times 10^6$ m^3) for power generation and the remaining 630,000 acre-ft (780×10^6 m^3) for irrigation. Essentially, all of the water diverted for power generation returns to the Colorado River. About 130,000 acre-ft (160×10^6 m^3) of the water diverted for irrigation are consumptively used by the crops, and most of the remainder returns to the rivers. Table 3 shows the annual irrigation diversions in Grand Valley and the quality of water available for application.

There is no ground water pumped into irrigation systems and there is no irrigation with ground water in the Grand Valley. Water recirculation and reclamation is practiced in several areas of the valley for reuse as irrigation supply. Return flows, administrative wastes, and field tailwaters are collected and used to supplement direct diversions on some private lands below the Grand

Valley Project. It is estimated that about 17,000 acre (6,900 ha) of land receive a portion of their irrigation water supply from these return flows, wastes, and tailwaters.

TABLE 2.—Gross Water Budget for Grand Valley

Source (1)	Flow, in acre-feet per year (2)
(*a*) Inflow	
Colorado River	2,700,000
Gunnison River	1,610,000
Plateau Creek	110,000
Intermittent stream	10,000
Precipitation	80,000
Total	4,510,000
(*b*) Losses Within Valley	
Consumptive use by crops	−130,000
Consumptive use by phreatophytes	80,000
Evaporation	20,000
Total	230,000
(*c*) Outflow From Valley	
Colorado River	4,280,000

Note: 1 acre-ft/yr = 1,233 m^3/yr.

TABLE 3.—Irrigation Diversions in Grand Valley[a]

Constituent (1)	Concentration (2)
Electroconductivity, in millimhos per centimeter	668
Total dissolved solids, in milligrams per liter	398
Suspended solids, in milligrams per liter[b]	400
Nutrients—nitrates, in milligrams per liter[c]	0.30
Nutrients—phosphates, in milligrams per liter[d]	0.05

[a] 1974–1976 average.
[b] Sediment, May–August 1951.
[c] Dissolved nitrate and nitrite as nitrogen.
[d] Dissolved ortho-phosphate.
Note: Quantity of flow (1971–1976 average) = 630,000 acre-ft/yr (770 × 10^6 m^3).

The quality of Colorado River water is excellent for irrigation in Grand Valley. With a TDS concentration that averages less than 400 mg/l and no other harmful materials, few problems have been experienced with its use.

Delivery and Irrigation Systems.—Irrigated lands in Grand Valley are served by three diversion dams and four major canal systems. Water is diverted from the Colorado River into the Government Highline and Orchard Mesa Canals

by the Grand Valley Diversion Dam above Cameo, Colorado. The Grand Valley Canal also obtains its water from the Colorado River with a diversion structure near Palisade, Colorado. Water for the Redlands Power Canal is diverted from the Gunnison River about 2 miles (3.2 km) above that river's confluence with the Colorado River. These canals and their diversion structures are shown on the map of Fig. 1. Table 4 shows these canals and a summary of their service.

The Government Highline Canal, the Orchard Mesa Canal, and their associated laterals make up the Grand Valley Project, which was constructed by the Water and Power Resources Service. The project serves about 38,200 acre (15,500 ha) in the northern and southeastern portions of the valley (called the Garfield Gravity and Orchard Mesa Divisions) and is administered by the Grand Valley Water Users Association. The Association is a local entity formed to operate and maintain the project facilities and repay project costs to the United States. The other canals and laterals in the valley are owned and operate by various private water user organizations. Most of the laterals, however, are owned and operated independently of the canals, and many of the owners have incorporated for operation and maintenance purposes.

TABLE 4.—Major Canal Systems of Grand Valley

Canal system (1)	Service area, in acres (2)	Initial capacity, in cubic feet per second (3)
Government Highline	28,200	875
Orchard Mesa	10,000	800
Grand Valley	28,800	640
Redlands Power	4,500	850
Total	71,500	—

Note: 1 ha = 2,471 acre; 1 cu ft/sec = 0.028 m^3 s.

Most of the canals, laterals, and farm ditches in the Grand Valley are open earth channels in contact with uncompacted alluvial soil or weathered Mancos Shale. Fewer than 20 mile (32 km) of the main canals in the valley are lined. In some areas, the canal and lateral banks are badly eroded and the channels enlarged, allowing considerable amounts of water to be lost through seepage.

Irrigation in Grand Valley is generally accomplished under water rights held by the entities which own, operate, and maintain the various canal and lateral systems. Nearly all of these rights are among the oldest of the active rights held along the Upper Colorado River. Because of these early priorities, Grand Valley irrigators are able to divert their full apportioned shares of water even in extremely dry years.

Gravity-furrow application systems are used for nearly all irrigation in Grand Valley. Direct turnouts, siphon tubes, small header ditches, etc., are prevalent in delivering water from the on-farm laterals to the crops in the fields. There is a small acreage of sprinkler irrigation in the valley, but it appears to be expanding very slowly. Gated pipe applications seems to be expanding rapidly. A 40-acre (16-ha) experimental plot of grapes and orchard is being irrigated

with a drip system, but the concept does not appear to be catching on in the valley.

Return Flows.—Return flows from Grand Valley irrigation are generated by four different mechanisms of the gravity irrigation systems: (1) Canal and lateral seepages; (2) deep percolation from on-farm use of water; (3) field tailwater runoff; and (4) administrative wastes from the canal and lateral systems. Water entering the ground-water system from canal and lateral seepage and on-farm deep percolation is either collected by natural drains that flow through the irrigated areas to the river or discharges diffusely into the Colorado River. Field tailwater and lateral wastewater are collected and reused widely for irrigation of lower lands in the valley, with only a portion of the water reaching the river directly. Administrative wastes from the canal systems normally discharge directly into a natural drain and are conveyed to the Colorado River. Table 5 shows the relative magnitude of flows from each of these sources and the influence of reuse of return flows upon discharges to the river.

TABLE 5.—Irrigation Return Flows in Grand Valley

Source or use (1)	Return Flow, in acre-feet per year		
	Surface (2)	Subsurface (3)	Combined (4)
Administrative waste	180,000		180,000
Field tailwater	190,000		190,000
Reuse for irrigation	−40,000		−40,000
Onfarm ditch infiltration		35,000	35,000
Canal seepage		50,000	50,000
Lateral seepage		50,000	50,000
On-farm deep percolation		35,000	35,000
Total	330,000	170,000	500,000

Note: 1 acre-ft/yr = 1,233 m^3/yr.

In the Grand Valley, any irrigation water that passes the root zone (whether from canal and lateral seepage or deep percolation) comes in contact with Mancos Shale, a highly saline formation that underlies most of the irrigated land in the valley. The percolating water dissolves salts and other minerals from the soil and underlying shale and carries them in solution to the Colorado River through drains and ground-water aquifers. It has been estimated that 650,000 ton–850,000 ton (590×10^6 kg–770×10^6 kg) of salt are added to the river annually in the valley. [This contribution averages about 780,000 ton/yr (710×10^6 kg/yr).]

Based on measurements of surface and ground-water flows and quality from 1974–1976, the annual salt contributions to the Colorado River from irrigation in Grand Valley are shown in Table 6. The average return flow TDS concentration calculated from water and salt balances generally agrees with actual, measured water quality data on Table 7.

The overall salinity levels of return flows in Grand Valley are influenced greatly by that portion of the flow that makes its way through the soil profiles and ground-water system. Wastewater, tailwater, and other surface contributions

pick up little dissolved material on their way to the drains. The ground-water contributions, however, increase their salinity levels over 10-fold. Table 7 shows measured salinity concentrations and other pertinent water quality data for these elements of the return flow.

There are no discharges into Grand Valley canals or laterals from municipalities or confined animal feeding operations. There is some overland flow into canals and laterals of the valley from lands above those features, but the volume is sporadic and rather insigificant, occurring mainly with summer thunderstorms.

TABLE 6.—Estimated Mean Annual Salt Contributions to Colorado River

Return flow source (1)	Return flow, in acre-feet per year (2)	Concentration, in milligrams per liter (3)	Salt load, in tons per year (4)
Canal seepage	50,000	3,500	240,000
Lateral seepage	50,000	3,500	230,000
Onfarm ditch infiltration	35,000	3,200	160,000
Onfarm deep percolation	35,000	3,200	140,000
Surface runoff from above irrigated land	—		10,000
Total			780,000

Notes: 1 acre-ft/yr = 1,233 m^3/yr; 1 ton/yr = 907 kg/yr.

TABLE 7.—Measured Water Quality Data for Grand Valley Return Flows

Constituent (1)	Return flow, concentration		
	Surface (2)	Subsurface (3)	Combined (4)
Electroconductivity, in millimhos per centimeter	650	6,800	2,700
Total dissolved solids, in milligrams per liter	400	4,100	1,400
Nutrients—nitrates, in milligrams per liter	[a]	[a]	4.0[b]
Nutrients—phosphates, in milligrams per liter	[a]	[a]	0.05[b]

[a]Not separable.
[b]Data from a 1-yr sampling program in 1976.

Receiving Waters.—The receiving water for return flows from irrigation in Grand Valley is the Colorado River between the community of Palisade and the Colorado/Utah border. The flow in this reach varies from 2,700,000 acre-ft/yr ($3,200 \times 10^6$ m^3) at Cameo on the upper end of the velley to about 4,280,000 acre-ft/yr ($5,280 \times 10^6$ m^3) at the lower end of the valley. Water depletions and contributions to the river in this area are shown in Table 2.

The salinity of the Colorado River upstream from Grand Valley is good,

having an average salt concentration of about 397 mg/L and an average annual salt load of about 1,650,000 ton ($1,497 \times 10^6$ kg). As the river traverses the valley, however, the quality of the water is degraded. At the Colorado/Utah border, the river has an average salinity concentration of about 571 mg/L and an annual salt load of about 3,760,000 ton ($3,410 \times 10^6$ kg), for a total increase of 2,110,000 ton ($1,910 \times 10^6$ kg). The Gunnison River, with a salinity concentration of about 654 mg/L, contributes about 1,330,000 ton ($1,210 \times 10^6$ kg) of this increase. The remaining increase of 780,000 ton (710×10^6 kg) annually results from irrigation return flows in the valley. This salinity increase and other water quality data for the Colorado and Gunnison Rivers are shown in Table 8.

TABLE 8.—Water Quality Data for Colorado and Gunnison Rivers

Constituent (1)	CONCENTRATION		
	Colorado River		Gunnison River
	Upstream end of Grand Valley[a] (2)	Downstream end of Grand Valley[b] (3)	Near Grand Junction (4)
Electroconductivity, in millimhos per centimeter	688	886	845
Total dissolved solids, in milligrams per liter	397	571	654
Calcium, in milligrams per liter	55	78	91
Magnesium, in milligrams per liter	13	27	31
Sodium, in milligrams per liter	66	71	61
Bicarbonate, in milligrams per liter	139	160	160
Sulfate, in milligrams per liter	97	245	312
Chloride, in milligrams per liter	89	58	10
Nitrate & Nitrite, in milligrams per liter	0.3	0.8	0.8

[a] United States Geological Survey station near Cameo, Colorado.
[b] Sampling station near Colorado/Utah border.

The 78,000 ton (71×10^6 kg) of salt contributed annually by irrigation in Grand Valley increase the salinity of the Colorado River by about 77 mg/L at Imperial Dam near Yuma, Ariz. As a result of this contribution and many other sources of salt, the river, which has historically had a high salt content, has been gradually increasing in salinity as flows of the river have been developed for man's use.

Salinity concentration of Colorado River streamflow progressively increases downstream as a result of water diversions and salt contributions from a variety of sources. In 1978, the average salinity concentration was 812 mg/L at Imperial

Dam, the last major diversion point in the United States. Unless control measures are undertaken, the concentration will continue to increase, reaching levels estimated by the Water and Power Resources Service at 1,150 mg/L–1,210 mg/L at Imperial Dam by the year 2000.

The high salt concentrations in the Lower Colorado River Basin adversely affect more than 14,000,000 people and about 1,000,000 acre (400,000 ha) of irrigated farmland in the United States. Affected most severely are municipal and industrial water users in the Los Angeles-San Diego area and irrigators in southern California and Arizona. According to a recent study by the Water and Power Resources Service, these users experience annual economic losses of $343,000 (in 1976 dollars) for each increase in salinity of 1 mg/L (in the range of 800 mg/L–1,000 mg/L) at Imperial Dam. These losses consist of $240,000 in damages to municipal and industrial water users, and $103,000 to irrigators. The losses associated with municipal and industrial water use occur primarily from increased water treatment costs, accelerated pipe corrosion and appliance wear, increased soap and detergent needs, and decreased drinking water palatability. For irrigators, the higher concentrations cause decreased crop yields, altered crop patterns, increased leaching and drainage requirements, and increased management costs.

Regulatory Programs that Affect Irrigation in Grand Valley

As previously described, return flow from irrigation in Grand Valley is a substantial contributor of salt to the Colorado River. There are three water quality legislative acts passed by the Congress of the United States that are of special importance to irrigation in Grand Valley: (1) The Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500); (2) the Colorado River Basin Salinity Control Act (Public Law 93-320); and (3) the Clean Water Act of 1977 (Public Law 95-217, which amended Section 208 of Public Law 92-500).

Public Law 92-500.—The basic objective of the Federal Water Pollution Control Act Amendments of 1972 is to restore and maintain the chemical, physical, and biological integrity of the Nation's waters. Its enactment introduced a new factor into the salinity problem of the Colorado River: it called for the establishment of numerical standards for salinity in the river.

The Colorado River Basin Salinity Control Forum was formed by the seven states of the Colorado River Basin (Colorado, Utah, Wyoming, New Mexico, Arizona, California, and Nevada) to establish water quality standards for salinity, including numeric criteria, for appropriate points in the Colorado River system. It also was charged to provide a plan to achieve compliance with these standards as expeditiously as practicable.

Consistent with Public Law 92-500, the Colorado River Basin Salinity Control Forum recommended flow-weighted average annual numeric salinity criteria for three locations in the lower main stem of the Colorado River system as follows:

Location	Salinity, in milligrams per liter
Below Hoover Dam	723
Below Parker Dam	747
At Imperial Dam	879

These criteria are a basinwide salinity standard and are fairly unique. They will allow for continued development of water resources and the cost-effective treatment of salinity contributions throughout the Colorado River Basin.

The water quality standards for salinity proposed by the Colorado River Basin Salinity Control Forum were approved by the Environmental Protection Agency and adopted by each of the basin states in 1976. The plan of implementation for these criteria comprises a number of Federal and non-Federal projects and measures to maintain the flow-weighted average annual salinity in the lower main stem at or below the recommended levels through 1990, as the basin states continue to develop their compact-apportioned waters. Key to this plan of implementation was the passage of the Colorado River Basin Salinity Control Act in 1974.

Public Law 93-320.—The economic detriments of salinity to water users in the basin and the need for some means to control salinity in the Colorado River led to the enactment of Public Law 93-320, the Colorado River Basin Salinity Control Act, in 1974. This legislation directed the Water and Power Resources Service to work with states of the basin and other Federal agencies to develop a program for improving water quality of the Colorado River. There are two main portions of Public Law 93-320—Title I and Title II.

Title I of the Colorado River Basin Salinity Control Act deals with the Colorado River below the Imperial Dam. Its main purpose is to enable the United States to comply with its obligations to Mexico under Minute No. 242 of the International Boundary and Water Commission, United States and Mexico. Construction of a desalting plant and complex near Yuma, Ariz., are the primary features of this part of the program to improve the quality of water delivered to Mexico.

The overall objective of Title II of Public Law 93-320 is to provide salinity control in the Colorado River while allowing the states to develop their compact apportioned shares of water from the river. Title II also has two main parts: (1) The construction of systems to control salinity contributions from four particular sources (Grand Valley, Paradox Valley, Las Vegas Wash, and Crystal Geyser); and (2) the expeditious completion of feasibility studies for treatment of 12 other known salt sources.

One of the specific provisions of Public Law 93-320 is the authorization for construction of the Grand Valley Unit of the Colorado River Basin Salinity Control Project. That unit will be directed toward reducing salinity contributions to the Colorado River associated with irrigated agriculture in the valley.

The Water and Power Resources Service plan for salinity control in Grand Valley is three-fold: (1) Line canals and laterals or place them in pipe to reduce or prevent seepage; (2) implement an Irrigation Management Services program to advise farmers on the timing and amounts of irrigation to minimize deep percolation; and (3) the Soil Conservation Service (SCS) onfarm facilities improvement program to minimize seepage and deep percolation in fields. These actions would prevent about 410,000 ton (370×10^6 kg) of salt from entering the river annually which would in turn reduce salinity of the river at Imperial Dam by 43 mg/L.

Under the previously mentioned program, 185 mile (298 km) of existing unlined canals in Grand Valley would be lined with concrete or other suitable materials, 450 mile (730 km) of laterals would be line or placed in pipe, and numerous structures would be replaced or added as needed. This part of the program

(called Water Systems Improvement) would account for about 280,000 ton (250 $\times$ 10^6 kg) annually of the reduction in salt contributed to the river.

Onfarm improvements of the SCS program would consist of lining some existing farm ditches, placing others in pipe, and combining them when possible. To increase the efficiency of land use, some land would be leveled and more efficient irrigation systems installed. These improvements, along with the Irrigation Management Services program, would account for about 130,000 ton (120 $\times$ 10^6 kg) annually of the reduction in salt contributed to the river.

Construction of salinity control facilities in Grand Valley is scheduled to begin during the fall of 1980. The reduction in salinity levels associated with those works [410,000 ton (370 $\times$ 10^6 kg) annually or 43 mg/L in concentration at Imperial Dam) is vital to the states efforts to meet water quality standards for the Colorado River.

Section 208.—Section 208 of Public Law 92-500 was amended by the Clean Water Act of 1977 to include return flows from irrigated agriculture in the nonpoint source classification rather than as a point source. This inclusion meant that areawide waste treatment management plans under Section 208 should include consideration of return flows and their cumulative effect. Grand Valley is part of the Colorado West designated 208 area and has been included in studies conducted by the Colorado West Area Council of Governments. The Preliminary Framework Plan of that agency was published in February, 1977.

The Water and Power Resources Service and SCS programs for salinity control in Grand Valley encompass the Section 208 recommendations of the above Preliminary Framework Plan. Implementation is the responsibility of the Water and Power Resources Service, Agricultural Stabilization and Conservation Service, and the local water user and irrigation companies. No additional work on irrigation return flows is included in Section 208 plans for the valley.

Conclusions

Irrigation in the Grand Valley of western Colorado contributes about 780,000 ton (708 $\times$ 10^6 kg) of salt annually to the Colorado River, causing an increase of about 77 mg/L in the salinity concentration at Imperial Dam. With water quality standards that have been established for the Colorado River and the continued development of water resources in the basin, a salinity control program for Grand Valley is needed. Water and Power Resources Service and SCS activities in the valley under Public Law 93-320 would meet the need for such a program and satisfy requirements for areawide wastewater management under Section 208 of Public Law 92-500.

Appendix.—References

1. "Alleviation of Salt Load in Irrigation Water Flow of the Upper Colorado River Basin," Final Report of Research Conducted by the Agricultural Research Service, and United States Salinity Laboratory, Sept., 1977.
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4. "Definite Plan Report, State One Development Grand Valley Unit, Colorado River

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WATER QUALITY CONTROLS ON IMPERIAL VALLEY DRAINAGE

By Vernon E. Valantine,[1] F. ASCE

INTRODUCTION

The Federal Water Pollution Control Act (FWPCA) as amended mandates the development of plans for the best management practices for irrigated areas that are to cover drainage flows. The ASCE Committee on Water Quality of the Irrigation and Drainage Division obtained Executive Committee approval for the formation of a Task Committee to analyze the water supply and drainage systems of several irrigation systems throughout the arid western United States. The Task Committee was also to consider the possible impacts of Federal water quality controls on the irrigation systems. This paper covers one of the irrigation systems analyzed by the Task Committee; the Imperial Irrigation District of California.

IMPERIAL IRRIGATION DISTRICT

The Imperial Irrigation District covers most of the irrigable lands of Imperial Valley, located in the southeastern corner of California. Imperial Valley, part of the delta region of the Colorado River, lies at and below sea level. The Valley is arid, with an annual rainfall of about 3 in. Maximum temperatures of 110° F–115° F (43° C–46° C) are not uncommon during the summer months, with monthly means of about 90° F (32° C). While frosts do occur, there are about 300 frost-free days a year.

Irrigation commenced in Imperial Valley in 1901, when water was diverted from the Colorado River near Yuma, Ariz., and transported through Mexico and back into the United States by means of an old bed of the Colorado River converted into a canal, called the Alamo Canal. During a major flood, the river broke through the diversion head works, flowed rampant into the Imperial Valley from 1905–1907, recreating the Salton Sea. In 1911, the District was organized to acquire the irrigation network of the California Development

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Note.—Discussion open until November 1, 1981. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on October 15, 1980. This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0233/$01.00.

Company that originally brought water to the Imperial Valley.

The All-American Canal became the District's water conveyance source upon its completion in 1942 by the United States Bureau of Reclamation, which is now the United States Water and Power Resources Service (WPRS). The WPRS schedules releases from the Colorado River reservoirs to meet the District's needs, while the All-American Canal (including its headworks at Imperial Dam) and distribution facilities are operated by the District.

The District delivers irrigation water to over 500,000 acre (200,000 ha), with approx 50,000 acre (20,000 ha) usually not being irrigated during a year. Of the land actively being cultivated, approx 20% is double cropped. The value of crops produced within Imperial Valley generally exceeds $500,000,000 a year. Vegetable and garden crops comprise about 13% of the harvested areas, permanent crops about 3%, and field crops comprise the balance of 84%. The leading field crops are cotton, alfalfa, grains, and sugarbeets.

Water Supply

The District's Colorado River water supply is diverted from the Colorado River at Imperial Dam and is conveyed by the unlined All-American Canal,

TABLE 1.—Water Quality Data Imperial Irrigation District Water Supply (year 1975)

Water quality parameters (1)	Annual averages (2)
Electroconductivity	1,300 μ-mhos
Total dissolved solids (salinity)	830 mg/L
Nutrients—nitrates	±2 mg/L
—phosphates	±0.02 mg/L
Pesticides	0.00 parts per trillion

about 80 miles (130 km) from Imperial Dam to the west side of Imperial Valley. In 1975, a fairly typical year, the District received 3,001,000 acre-ft (3,705,000 m^3) at Drop No. 1 on the All-American Canal, just below the point where the Coachella Valley County Water District's water supply leaves the Canal. There is essentially no ground water pumped within the District due to the high salinity of the ground water underlying the valley floor and low yields of any wells. Some individual farmers are recapturing excess irrigation runoff and reusing it, but the present volume of water so captured and reused within the District is very small. The only feasible source of water for the District is importation from the Colorado River.

Table 1 lists water quality data for the District's diversions from the Colorado River for 1975. As shown by the data in Table 1, Colorado River water diverted by the District has high electroconductivity and total dissolved solids levels, but has a very low nutrient and pesticide content.

Delivery and Irrigation System

The District's irrigation system consists of long main canals running generally north from the All-American Canal through Imperial Valley, with laterals and

sublaterals serving the farmers and cities in the Valley. The canals and laterals were originally constructed in earth sections, but the District has been concrete-lining its canals and laterals gradually since 1954. As of 1978, the District had lined about 690 miles (1,110 km) and was continuing to line the remaining 780 miles (1,260 km) of lateral canals.

Each farmer may either use the canal water directly for flood and furrow irrigation with no input of energy, or may install pumps to deliver water by sprinkler to his crops. Most lands in Imperial Valley are irrigated by flood or furrow methods. Because of the type of soils prevalent throughout the Valley, coupled with the relatively high salinity level of Colorado River water, vegetable crops are generally sprinkled during the preemergence period of crop development, with furrow irrigation being relied upon to bring the crops to maturity. Sprinklers are the permanent system for water application on only a small area in the Valley.

Drainage Flows

The Imperial Valley soils are predominately fine-grained, are slow draining, and are underlain by highly saline ground waters close to the ground surface. In order to maintain the productivity of the Valley's lands, the District has constructed a network of open drainage ways and has vigorously supported the efforts of local farmers to install tile lines under their fields, discharging into these open drains. Most of the open drains discharge, in turn, into the New and Alamo Rivers, which are international streams by definition. These streams convey both surface runoff from irrigation and subsurface drainage flows to the Salton Sea. In 1975, tile drains discharged approx 324,000 acre-ft (400,000,000 m^3). In addition, other drainage flows, originating as irrigation runoff and subsurface flow to the open drains, totaled about 440,000 acre-ft (543,000,000 m^3). These flows are generated by the irrigation systems in Imperial Valley.

Imperial Valley has a major livestock feeding industry, with about a half-million cattle and a few hundred-thousand sheep on feedlots and pasture during a typical year. Manure is stockpiled and other feedlot wastes are contained on the premises to prevent wastes from entering the drainage network of the Valley. As the New and Alamo Rivers act as the drainage system for the entire Valley, treated sanitary sewage effluent is discharged into the rivers from nine communities serving a population of over 64,000. The municipalities also rely upon surface drainage ditches to carry away any rainfall runoff.

The Salton Sea is the receiving body of water for the return flows from irrigation of District lands, waste flows from the municipalities of Imperial Valley, and both irrigation and municipal drainages and wastes of the Mexicali Valley of Mexico. During the 5-yr period, 1967–71, the annual drainage flows into the Sea averaged 1,107,000 acre-ft (1,367,000 m^3), with about 100,000 acre-ft (123,000,000 m^3) of the total originating in Mexico. The salinity or total dissolved solids levels in the drainage flows were subtantially higher than in the supply water, at about 2,600 mg/L, and the nutrient content was also up substantially, at 5.8 mg/L of nitrates and 0.4 mg of phosphates. The source of the dissolved solids is the imported irrigation water, and its concentration is a function of the degree of irrigation efficiency achieved in the District. As the District is

aggressively undertaking measures to increase overall efficiencies, from the lining of canals and laterals to the adoption of both incentive and penalty programs applicable to individual farmers to improve their efficiencies, the salinity of the drainage water will tend to increase in the future.

Receiving Water

The receiving water for drainage from the District lands is the Salton Sea, which has been legislatively defined as an agricultural sink by both federal and state laws. Being a terminal body of water, the Sea's level rises and falls over time, depending upon varying levels of inflow to the Sea. During the last several years, the watershed draining into the Sea has experienced heavy deluges each year. This unusual precipitation pattern and the resulting high runoff has caused the Sea's surface to rise several feet, to about 226 ft (69 m) below sea level. At a surface elevation below the mean sea level, with a storage capacity and surface area of −220 ft (67.1 m), the surface area is 262,300 acre (106,150 ha), and the volume is 8,996,000 acre-ft (11,110,000 m^3). At −230 ft (−70.1 m), the surface area is 235,600 acre (95,350 ha), and the volume is 6,506,000 acre-ft (8,030,000 m^3).

TABLE 2.—Salton Sea Water Quality Data

Water quality parameters (1)	Annual averages (2)
Electroconductivity	48,000 μ-mhos
Total dissolved solids (salinity)	39,000 mg/L
Nutrients—nitrates	3.3 mg/L
—phosphates	0.1 mg/L
Pesticides	10 parts per trillion

Table 2 gives the water quality characteristics of the Salton Sea. The nutrient level of the Sea, continually being replenished by the drainage water inflows, supports a flourishing ocean water fishery that was introduced into the Sea. The salinity of the Sea exceeds that of the ocean at the present time, and until the last few years, has been slowly rising. The Sea's rising salinity threatens the reproductive ability of the game fish species within the Sea. Because of the salinity of the New and Alamo Rivers is only about one-fifteenth the Sea's salinity, drainage inflows slows down the Sea's increase in salinity.

Summary and Conclusions

Agriculture in the arid lands covered by the District is dependent upon the importation of Colorado River water for irrigation. The continuance of the productive usability of these lands is further dependent on the removal by drainage of the salts brought into the Valley in the irrigation water. Without adequate drainage, the salts in the soils would have rapidly increased under irrigation to the point that lands would have become unproductive, duplicating world-wide patterns of boom and bust desert irrigation experiences.

Because of Imperial Valley's location in a closed basin at and below sea level, its drainage flows cannot be discharged to the ocean and must be stored within the basin. Accordingly, the basin's sump, the Salton Sea, serves the essential function of the receiving body for the Valley's drainage flows, and the continuance of agriculture within the Valley is dependent upon the continued dedication of the Sea for that function.

Levels of total dissolved solids (salinity) in the District's drainage water are not susceptible to reduction through modifications in irrigation practices, except through dilution of the drainage flows through lowered efficiencies or other means. As lower irrigation efficiency is unacceptable in a water-short area, it is concluded that the control of salinity in drainage flows or the Salton Sea should not be an objective in any best management practices that may be developed pursuant to the FWPCA for the Imperial Valley.

Since the primary purpose of the receiving waters is to receive the salt load removed from the soil profile under irrigation, and since all other possible uses of the waters are incidental to their carrying out this function of salt removal, the imposition of any controls on this function would tend to defeat this purpose. Accordingly, it is concluded that best management practices for return flows from the District lands should be based on continuation of the removal and transfer of dissolved salts from irrigated lands to the Salton Sea.

DISCUSSION

Note.—This paper is part of the Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. IR2, June, 1981. ISSN 0044-7978/81/0002-0241/$01.00.

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Runoff Curve Numbers from Partial Area Watersheds[a]

Closure by Richard H. Hawkins,[4] M. ASCE

Smith and Montgomery have raised several points of misunderstanding and clarification. Insofar as possible, these will be treated in order.

Rational Formula or Curve Number.—The reader is indeed "examining the venerable 'rational formula' " in the context of how it appears in a curve number disguise. Curve numbers are widely used, and bear the imprimatur of the U.S. Department of Agriculture, while the rational formula is considered to be poor taste. The message of the paper is to look for a "rational formula" structure whenever curve numbers shrink with increasing storm depth. An accompanying obvious message is that the CN method doesn't agree too well with reality, at least for the cases studied.

Physical Evidence of *C*.—Neither direct nor remotely-sensed measurements of the wet and impervious fractions were made. The data were all taken from available sources which omitted detailed watershed characteristics. However, within the past year, a source area survey of two of Davis County watersheds was undertaken by Pankey (17), and her results are given in Table 3, with comparisons to the C values given in Table 2. In approximate terms, C is twice the impervious area fraction measured by Pankey. Though it may be no more than a curious coincidence, a similar finding was made by Betson (16) who stated that, "Most of the storm runoff results from an area about twice the size of the swampy portion of the watershed." in dealing with the performance of some small watersheds in the southeastern U.S.

Application Range.—Smith and Montgomery raise an interesting issue when they claim, ". . . the data used by the author fall into the small region in Fig. 1 where $P/S < 0.4$, which poorly (if at all) defines a CN in any case . . ." The writer is not aware of any stated or speculated constraints in the open CN literature on the application range, except that $P/S > 0.2$. From the data sets used, values of P/S in the 0.4 range are apparently natural, expected, and routine in the geographical conditions sampled. The discussion of rainfall depth perspective (p. 387) dealt with this gray area. The conventional application of CN method in the eastern humid agricultural situations matches high storm depths and high CNs, and high P/S. With western wildlands, low rainfalls and low CNs prevail, thus low P/S. Where *is* the threshold of application?

Data Censoring.—Since no points fell on the CN_o line, were the $Q = 0$ storms omitted? It is not known that this did (or did *not*) occur, although the data sets are not complete summaries of all summer storms. The question, however, is quite appropriate, and should be raised whenever storm rainfall-storm runoff studies are made. For example, the original development of the CN method

[a]December, 1979, by Richard H. Hawkins (Proc. Paper 15024).

[4]Prof., Dept. of Forestry and Outdoor Recreation (Watershed Science Unit), and Dept. of Civ. and Environmental Engrg., Utah State Univ., Logan, Utah 84322.

arose from a large body of P-Q data of unknown pedigree and selection, and thus the same issue might be raised.

Similarly, as pointed out, Eq. 8 should bear the constraint of $Q > 0$. At $Q = 0$, it will calculate CN_o, which would require $S \geq 5P$. Zero runoff has meaning only when $S = 5P$, or when I_a has just been satisfied.

Alternate Explanations.—The use of an alternate model, drawing on infiltration and intensity reasoning, illustrates that the runoff and curve number behavior shown in Figs. 4–7 could well have arisen through other processes. The illustration does not cope with the confirmed existence of an unknown area of impervious flowing water. The two notions in combination might produce even more justifiable and agreeable results. For example, runoff from Ephriam watershed A (Fig. 10) is sensitive to extreme intensities. The observed shortfall between the measured impervious fraction and fitted C (item 2 previously, and Table 3) also encourages a search for additional runoff sources. At the very least, the

TABLE 3.—Results from Source Survey of Davis County Watersheds

Watershed (1)	C from Table 2 (2)	Source fraction,* from Pankey (17) (3)
West Chicken Creek	0.0075	0.0045
Halfway Creek	0.0105	0.0058

*Fraction of the watershed in springs, rock outcrops, ponds, active channels, and adjacent small impervious areas.

alternate source argument contributes another explanatory argument against acceptance and usage of the CN method.

Appendix.—References

16. Betson, R. P., "What is Watershed Runoff? *Journal of Geophysical Research*, Vol. 69, 1964, pp. 1541–1552.
17. Pankey, J. M., "Field Estimates of Impervious Areas of Two Partial Area Watersheds," thesis presented to Utah State University, at Logan, Utah, in 1980, in partial fulfillment of the degree of Master of Forestry.

Erosivity Values for Individual Design Storms[a]

Discussion by George R. Foster[2]

The author developed regression equations for estimating maximum 30-min intensity and erosivity for Soil Conservation Service design storms. These equations can be improved by a more detailed analysis of storm energy.

Where rainfall hyetographs are smooth, continuous, single-peaked functions like the SCS curves, total storm energy E can be calculated from:

$$E = \int_0^D e\, i\, dt \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which e = rainfall energy per unit rainfall; i = rainfall intensity as a function of time; t = time; and D = storm duration. Substituting the author's Eq. 1 for e in Eq. 8 gives:

$$E = 916\,P + 331 \int_0^D i \log i\, dt \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which P = amount of rainfall in the storm. After normalization of intensity and time by $i_* = iD/P$ and $t_* = t/D$, Eq. 9 becomes:

$$E = P\left[916 + 331 \log\left(\frac{P}{D}\right) + 331 \int_0^1 i_*\, \log i_*\, dt_*\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

since $\int_0^D i dt = P$ and $\int_0^1 i_*\, dt_* = 1$. The sum in brackets gives average energy, $\bar{e}$, per unit rainfall for the storm. The term:

$$e_{\bar{i}} = 916 + 331 \log\left(\frac{P}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

gives energy per unit rainfall based on the storm's average intensity (P/D). The term

$$e_n = 331 \int_0^1 i_*\, \log i_*\, dt_* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

accounts for the nonuniformity of rainfall intensity and depends only on the normalized distribution of intensity and not on volume or duration. Eq. 12 was numerically integrated for the SCS curves. The result is exact, except for errors due to numerical integration that can be made as small as desired. Values for e_n are given in Table 6. Therefore, Eq. 10 and Table 6 are a means of almost exactly computing energy for the SCS design curves.

[a]June, 1980, by Keith R. Cooley (Proc. Paper 15462).

[2]Hydr. Engr., U.S. Dept. of Agric., Science and Education Administration, Agricultural Research, Dept. of Agricultural Engrg., Purdue Univ., Lafayette, Ind.

The author's Eq. 2 for maximum 30-min intensity can be rewritten as

$$I = \left(\frac{P}{D}\right)(\alpha' D)^{1+\beta} \quad \ldots \quad (13)$$

in which α' = a constant to be determined. When $D = 0.5$ h, I must equal P/D, which can only occur if $\alpha' = 2$. Therefore, Eq. 13 can be rewritten as:

$$I = \left(\frac{P}{D}\right)(2D)^{\gamma} \quad \ldots \quad (14)$$

in which γ is evaluated from the author's β as $1 + \beta$. Values for γ are given in Table 7. Eq. 14 eliminated the author's parameter α.

TABLE 6.—Factor Accounting for Nonuniform Distribution of Rainfall for Computing Storm Energy for SCS Design Storms

Storm type (1)	Nonuniformity factor e_n (2)
IA	28
I	65
II	142
IIA	257

TABLE 7.—Exponent for Equation for Maximum 30-min Intensity

Storm type (1)	Exponent (2)
IA	0.44
I	0.60
II	0.75
IIA	0.864

Combining Eqs. 10, 12, and 14 gives an equation for EI that is very accurate, assuming that the author's equation for maximum 30-min intensity is very accurate:

$$EI = P^2\left[916 + 331 \log\left(\frac{P}{D}\right) + e_n\right] 2^{\gamma} D^{\gamma - 1} \quad \ldots \quad (15)$$

Storm erosivity can also be written as:

$$EI = \bar{e} P I \quad \ldots \quad (16)$$

Assuming that average unit energy, $\bar{e}$, for a storm is a power function of average intensity, P/D, for the storm gives:

$$\bar{e} = \mu\left(\frac{P}{D}\right)^{\xi} \quad \ldots \quad (17)$$

which substituted in Eq. 1 gives:

$$EI = \frac{\mu 2^{\gamma} P^{2+\xi}}{D^{\xi-\gamma+1}} \quad \ldots \ldots \ldots

in this order. Although curves can be compared over the entire time duration their critical point is the normalized time, 30 min divided by storm duration. For example, the 30-min normalized time for the 3.53 h, August 17, 1957 storm is 0.14. At this point, the observed rainfall curve in Fig. 5 is above the Type-IIA curve, indicating that actual *EI* is at least as great as that estimated from a Type-IIA curve. The September 10, 1967 observed rainfall curve suggests an *EI* value slightly less than that for the Type-II curve. The actual value is about halfway between the values for Type-I and Type-II curves. The July 24, 1972 curve suggests an *EI* value slightly above that halfway between values for Type-I and Type-II curves. The actual *EI* is slightly above the *EI* for a Type-I curve. This shows that data from observed rainfall having a single peak intensity can be rearranged to allow visual selection of an SCS curve to compute *EI*.

This discussion was written in cooperation with the Purdue Agricultural Experiment Station (*Purdue Journal No. 8218*).

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Original papers should be submitted in triplicate to the Manager of Technical and Professional Publications, ASCE, 345 East 47th Street, New York, N.Y. 10017. Authors must indicate the Technical Division or Council, Technical Committee, Subcommittee, and Task Committee (if any) to which the paper should be referred. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

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